STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

FOUNDATIONS AND SUBSTRUCTURE

PART-1: GENERAL FEATURES AND WORKING STRESS DESIGN

(Third Revision)

INDIAN ROADS CONGRESS
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CONTENTS

S. No.	Description	Page No.
Personnel of the Bridge Specifications and Standards Committee		i-ii
Background/Introduction		1
700	**Scope**	3
701	**Terminology**	3
	701.1 Abutment	3
	701.2 Afflux	3
	701.3 Balancer	3
	701.4 Bearing Capacity	3
	701.5 Bearing Pressure	4
	701.6 Cofferdam	4
	701.7 Foundation	4
	701.8 Pier	4
	701.9 Piles	4
	701.10 Retaining Wall	5
	701.11 Substructure	6
	701.12 Well Foundation	6
702	**Notations**	6
703	**Discharge and Depth of Scour for Foundation Design**	8
	703.1 Design Discharge of Foundation	8
	703.2 Mean Depth of Scour	8
	703.3 Maximum Depth of Scour for Design of Foundation	9
704	**Sub-surface Exploration**	10
	704.1 Objectives	10
	704.2 Zone of Influence	10
	704.3 Methods of Exploration	11
	704.4 Geophysical Investigations	11
705	**Depth of Foundation**	11
	705.1 General	11
	705.2 Open Foundations	11

	705.3	Well Foundations	12
	705.4	Pile Foundations	13
706	**Loads, Forces, Stability and Stresses**		13
	706.1	Loads, Forces and their Combinations	13
	706.2	Horizontal Forces at Bearing Level	14
	706.3	Base Pressure	14
707	**Open Foundations**		16
	707.1	General	16
	707.2	Design	16
	707.3	Open Foundations at Sloped Bed Profile	16
	707.4	Construction	16
	707.5	Raft Foundations	17
708	**Well Foundations**		17
	708.1	General	17
	708.2	Well Steining	18
	708.3	Design Considerations	19
	708.4	Stability of Well Foundations	20
	708.5	Tilts and Shifts	21
	708.6	Cutting Edge	21
	708.7	Well Curb	21
	708.8	Bottom Plug	22
	708.9	Filling the Well	22
	708.10	Plug over Filling	22
	708.11	Well cap	22
	708.12	Floating Caissons	23
	708.13	Sinking of Wells	23
	708.14	Pneumatic Sinking of Wells	23
	708.15	Sinking of Wells by Resorting to Blasting	24
709	**Pile Foundation**		24
	709.1	General	24
	709.2	Requirement and Steps for Design and Installation	26
	709.3	Geotechnical Capacity of Pile	28

709.4	Structural Design of Piles ..... (Deleted)	31
709.5	Design of Pile Cap ..... (Deleted)	31
709.6	Important Consideration, Inspection/Precautions for Different Types of Piles	31
710	**Substructure**	32
710.1	General	32
710.2	Piers	33
710.3	Wall Piers ..... (Deleted)	34
710.4	Abutments	34
710.5	Abutment Pier	35
710.6	Dirt Walls, Wing Walls and Return Walls	35
710.7	Retaining Walls	36
710.8	Pier and Abutment Caps	36
710.9	Cantilever Cap of Abutment and Pier ..... (Deleted)	37
710.10	Pedestals below Bearing	37
Appendices		
Appendix-1	**Guidelines for Sub-Surface Exploration**	38
Appendix-2	**Guidelines for the Design and Construction of Raft Foundation for Road Bridges**	53
Appendix-3	**Procedure for Stability Calculation**	65
Appendix-4	**Precautions to be taken during Sinking of Wells**	68
Appendix-5	**Capacity of Pile based on Pile Soil Interaction**	72
Appendix-6 Part-1	**Method – 1: Pile Load Capacity by Dynamic Test using Wave Equation**	78
	Method – 2: Pile Dynamic Test Method based on Hiley's Formulae (By Laser/infrared Operated Equipment)	82
Appendix-6 Part-2	**Standard Test Method for Low Strain Pile Integrity Testing**	85
Appendix-7	**Bi-Directional Load Testing of Piles**	88
Appendix-8	**Filling behind Abutments, Wing and Return Walls**	105
Appendix-9	**Guidelines for Calculating Silt Factor for Bed Material Consisting of Clay**	107

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(as on 5th June, 2022)

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2	Director General (Road Development) & Special Secretary to Govt. of India	Ministry of Road Transport and Highways, New Delhi
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FOUNDATIONS AND SUBSTRUCTURE

BACKGROUND

The "Standard Specifications and Code of Practice for Road Bridges Section VII Foundations and Substructure" was first published in July 1980 as Part I — General Features of Design. Later first revision was published in December, 1983 incorporating Part II and Amendments 1,2 and 3 to Part I as Unified Code. The second revision of this code was undertaken by the Foundation and Structure Committee (B-4) and the initial draft was finalized by the Committee under the Convenorship of Shri R.H. Sarma. Subsequently, the draft was reconsidered and discussed in various meetings by the reconstituted Foundation, Sub-structure and Protective Works Committee (B-4) under Convenorship of Shri S.A Reddi. The final draft as approved by BSS Committee was subsequently approved by the Executive Committee in its meeting held on 30.8.2000. It was later approved by the Council in its 160th meeting held at Kolkata on 04.11.2000 for publishir.g the revised IRC Code Section VII: IRC: 78:2000. After that numerous amendments and errata were published to this Code based on development in design and construction technology.

Subsequently, the Revised Edition of IRC:78 "Standard Specifications and Code of Practice for Road Bridges" Section VII — Foundation incorporating all amendments and Errata published till 2013 was approved by Foundations and Substructure Foundation Substructure Protective Works and Masonry Structures Committee (B-3) in its meeting held on 16.10.2013. The Revised Edition of IRC:78 was approved by the Bridges Specifications and Standards Committee in its meeting held on 06.01.2014 and Executive Committee held on 09.01.2014 for publishing.

The IRC B-3 Committee took up further revision of this code based on the direction of BSS Committee to transfer the Clauses pertaining to discharge and scour to IRC:5 and Structural Design to IRC:112. Simultaneous other clauses were reviewed. The Committee also formulated Code of Practice for design of foundations on Limit State Approach. The Code has been numbered as IRC:78 Part 2. Accordingly this revised code has been numbered as IRC:78 Part 1.

The IRC B-3 Committee deliberated in a number of meetings and finalized the revised code in its meeting held on 02.05.2022 and recommended its submission to Bridge Specifications and Standards Committee (BSS) for its consideration.

The Composition of B-3 Committee is given below:

Sharma, R.S.	.....	Convenor
Basa, Ashok	.....	Co-Convenor
Mayur, P.V.	.....	Member-Secretary

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Secretary to Govt. of India

Ministry of Road Transport & Highways

Secretary General,
Indian Roads Congress

(Nirmal, S.K.), Additional Director
General-Nodal, Ministry of Road
Transport and Highways

The revised code was approved by the Bridge Specifications and Standards Committee (BSS) in its meeting held on 05.06.2022 and subsequently by the Executive Committee (EC) in its meeting held on 16.06.2022. The revised code was approved by the IRC Council in its 222nd meeting held from 17th to 18th June, 2022 at Shillong (Meghalaya) for printing.

700 SCOPE

This code deals with the design and construction of foundations and substructure for road bridges including viaducts, culverts, grade separated structures and rail road bridges. The provisions of this code are meant to serve as a guide to both the design and construction engineers, but mere compliance with the provisions stipulated herein will not relieve them in any way of their responsibility for the stability and soundness of the structure designed and erected.

701 TERMINOLOGY

The following definitions shall be applicable for the purpose of this code.

701.1 Abutment

The end support of the deck (superstructure) of a bridge, which also retains earth, fill of approaches behind fully or partly.

701.1.1 *Box type abutment and return wall*

When the return walls on two sides are integrated with abutment and a back wall parallel to abutment is provided at the end of returns with or without additional internal wall along or across length, this structure is called box type abutment and return wall, or end block.

701.1.2 *Non-load bearing abutment*

Abutment, which supports the end span of less than 5 m.

701.1.3 *Non-spill through abutment*

An abutment structure where the soil is not allowed to spill through.

701.1.4 *Spill through abutment*

An abutment where soil is allowed to spill through the gaps of columns placed along the length of abutment, or where columns are placed below deck beams and gap in between is free to spill earth. (Spilling of earth should not be permitted above a level of 500 mm below the bottom of bearings).

701.2 Afflux

The rise in the flood level of the river immediately on the upstream of a bridge as a result of obstruction to natural flow caused by the construction of the bridge and its approaches.

701.3 Balancer

A bridge/culvert like structure provided on embankment to allow flow of water from one side of the embankment to other side, for purpose of avoiding heading up of water on one side or for avoiding blocking the entry to the other side.

701.4 Bearing Capacity

The supporting power of soil/rock is referred to as its bearing capacity.

701.4.1 *Allowable bearing pressure*

It is the maximum gross pressure intensity at which neither the soil fails in shear, (after accounting for appropriate factor of safety) nor there is an excessive settlement beyond permissible limits, which is detrimental to the structure.

701.4.2 *Net safe bearing capacity*

It is the net ultimate bearing capacity divided by a factor of safety as per Clause 706.3.1.1.

701.4.3 *Net ultimate bearing capacity*

It is the minimum net loading intensity causing shear failure of the soil.

701.4.4 *Gross Safe bearing capacity*

The maximum intensity of loading, which the soil can carry safely without risk of shear failure and it is equal to the net safe bearing capacity plus original overburden intensity.

701.4.5 *Gross Ultimate gross bearing capacity*

It is the minimum gross load intensity at the base of the foundation at which the soil fails in shear.

701.5 Bearing pressure

701.5.1 *Gross Pressure intensity*

It is the total pressure at the base of the foundation on soil due to the possible combinations of load and the weight of the earth fill.

701.5.2 *Net pressure intensity*

It is the difference in intensities of the gross pressure and the original overburden pressure.

701.6 Cofferdam

A structure temporarily built for the purpose of keeping water or soil sufficiently away to permit construction or proceed without excessive pumping and to support the surrounding ground.

701.7 Foundation

The part of a bridge in direct contact with founding strata and transmitting load to it. In general, the structure below the general ground level or bed level for open foundations, below pile cap with pile foundation and below well cap for structures with well foundations are referred as foundation. A clarity is required to distinguish between foundation and substructure.

701.8 Pier

Intermediate supports of the deck (superstructure) of a bridge.

701.8.1 *Intermediate Heavy Pier*

Generally used in multiple span arch bridges. Intermediate heavy pier is designed for a condition that even if one side arch span collapses it would be safe. These are provided after three to five spans.

701.8.2 *Abutment Pier*

The abutment on which the soil pressure is taken by reinforced wall is referred as abutment pier.

701.9 Piles

701.9.1 *Bearing piles or friction piles*

A pile driven or cast-in-situ for transmitting the weight of a structure to the founding strata by the resistance developed at the pile base and by friction along its surface. If it supports the load mainly by the resistance developed at its base, it is referred to as an end-bearing pile, and if mainly by friction along its surface, it is referred to as a friction pile.

701.9.2 *Bored cast-in-situ pile*

A pile formed with or without a casing by boring a hole in the ground and subsequently filling it with plain or reinforced concrete.

701.9.3 *Driven cast-in-situ pile*

A pile formed in the ground by driving a permanent or temporary casing, and subsequently filling it with plain or reinforced concrete.

701.9.4 *Driven pile*

A pile driven into the ground by the blows of a hammer by a vibrator.

701.9.5 *Precast pile*

A reinforced or prestressed concrete pile cast before driving, or installing in bore and grouted.

701.9.6 *Raker or batter pile*

A pile installed at an inclination to the vertical.

701.9.7 *Sheet pile*

One or a row of piles driven or formed in the ground adjacent to one another in a continuous wall, each generally provided with a connecting joint or interlock, designed to resist mainly lateral forces and to reduce seepage; it may be vertical or at an inclination.

701.9.8 *Tension pile*

A pile subjected to tension/uplift is called tension pile.

701.9.9 *Test pile*

A pile to which a load is applied to determine and/or to confirm the load carrying capacity (ultimate load/working load) of the pile and the surrounding ground.

701.9.10 *Working pile*

One of the piles forming the foundation of the structure.

701.10 Retaining Wall

A wall designed to resist the pressure of earth filling behind.

701.10.1 *Return wall*

A wall adjacent to abutment generally parallel to road or flared up increase the width and raised up to the top of road.

701.10.2 *Toe wall*

A wall built at the end of the slope of earthen embankment to prevent slipping of earth and/ or pitching on embankment.

701.10.3 *Wing wall*

A wall adjacent to abutment with its top upto road top level near abutment and sloping down upto ground level or a little above at the other end. This is generally at 45° to the alignment of road or parallel to the river and follows profile of earthen banks.

701.11 Substructure

The bridge structure, such as, pier and abutment above the foundation and supporting the superstructure. It shall include returns and wing walls but exclude bearings.

701.12 Well Foundation

A type of foundation where a part of the structure is hollow, which is generally built in stages or lifts through ground or water to the prescribed depth by removing earth through dredge hole.

701.12.1 *Till of well*

The inclination of the axis of the well from the vertical expressed as the tangent of the angle between the axis of the well and the vertical.

701.12.2 *Shift of well*

The horizontal displacement of the centre of the well at its base in its final position from its designed position

702 NOTATIONS

For the purpose of this code, the following notations have been adopted:

A_1 Dispersed concentric area

A_2 Loaded area

B Width between outer faces of pile group in plan parallel to the direction of movement

C The allowable bearing pressure with near uniform distribution on the founding strata

C_o The permissible direct compressive stress in concrete at the bearing area of the base

D Diameter of pile

D_b Discharge in cubic metre/sec, (cumecs) per metre width

d External diameter of circular well in metre

d_m Weighted mean diameter in mm of bed material

d_{sm} Mean depth of scour in metre below flood evel

F_b Longitudinal force due to braking

F_{cf} Centrifugal force

F_d Deformation effects

F_h Horizontal force

F_{ep} Earth pressure

F_{eq} Seismic force

F_{er} Erection effects

F_f Frictional force at bearings

F_{Im} Impact due to floating bodies

F_s Secondary effects

F_{wc} Water Current

F_{te} Temperature effects [See Note (i)]

F_{wp} Wave pressure [See Note (ii)]

G Dead load

G_b Buoyancy

G_b Snow load

h Minimum thickness of steining in metre

K_a Co-efficient of active earth pressure

K_p Co-efficient of passive earth pressure

K_{sf} Silt factor

L Length between outer faces of pile group in plan parallel to the direction of movement

l_{te} Movement of deck over bearings, other than due to applied force

l Depth of well

l_s Depth of well in metre, up to MSL.

N Standard penetration test value

P_a Total active pressure

P_p Total passive pressure

Q Live load

R_g Dead load reaction

R_q Live load reaction

V_r Shear rating of elastomeric bearing

W Wind load

α Reduction factor.

ß Ratio of long side to the short side of the footing

S_u Undrained shear strength

q_u Undrained cohesion

μ Co-efficient of friction

ϕ Angle of internal friction

δ Settlement of pile

δ_g Settlement of pile group

NOTES: i) Temperature effects (F_{te}) in this context is not the frictional force due to the movement of bearing but that which is caused by rib shortening, etc.

ii) The wave forces shall be determined by suitable analysis considering drawing and inertia

forces, etc., on single structural members based on rational methods or model studies. In case of group of piles, piers, etc., proximity effects shall also be considered.

703 DISCHARGE AND DEPTH OF SCOUR FOR FOUNDATION DESIGN

703.1 Discharge for Foundation Design

703.1.1 To provide for an adequate margin of safety, the scour for foundation design shall be calculated for a larger discharge over the design discharge determined as per IRC: 5 as given below:

Catchment area in km^2	Increase over design Discharge in percent
0-3000	30
3000-10000	30-20
10000-40000	20-10
Above-40000	10

Note: i) For intermediate values of catchment area, linear interpolation may be adopted.

ii) The minimum vertical clearance above the HFL already determined as per IRC: 5 need not be increased

703.2 Mean Depth of Scour

$$d_{sm} = 1.34\left(\frac{D^2_b}{K_{sf}}\right)^{1/3}$$

The mean scour depth below Highest Flood Level (HFL) for natural, channels flowing over scourable bed can be calculated theoretically from the following equation:

Where

D_b= The design discharge for foundation per metre width of effective water way.

K_{sf}= Silt factor for a representative sample of bed material obtained upto the level of anticipated deepest scour.

703.2.1 The value of D_b may be determined by dividing the design discharge for foundation by lower of theoretical and actual effective linear waterway as given in IRC: 5.

703.2.2 'K_{sf}' is given by the expression $1.76\sqrt{d_m}$, d_m being the weighted mean diameter in millimetre.

703.2.2.1 The value of K_{sf} for various grades of sandy bed are given below for ready reference and adoption:

Type of bed material	d_m	K_{sf}
Coarse silt	0.04	0.35
Silt/fine sand	0.081 to 0.158	0.5 to 0.7
Medium sand	0.223 to 0.505	0.85 to 1.25
Coarse sand	0.725	1.5
Fine bajri and sand	0.988	1.75
Heavy sand	1.29	2.0 to 2.42

For boulder strata the values of d_m and K_{sf} may be taken from IRC:5.

703.2.2.2 No rational formula or data for determining scour depth for bed material consisting of clayey bed is available. In absence of any data on scour for such material, the meanscour depth may be calculated following the guidelines given in **Appendix-9**.

703.2.3 If there is any predominant concentration of flow in any part of waterway due to bend of the stream in immediate upstream or downstream or for any other reason, like, widevariation of type of bed material across the width of channel, then mean scour depth may be calculated by dividing the waterway into compartments as per the concentration of flow.

703.2.4 In case of bridge mainly adopted as balancer, the mean scour depth 'd_{sm}' maybe taken as (Highest Flood Level-Lowest Bed Level) divided by 1.27.

703.2.5 Scour depth may be determined by actual observations wherever possible. This is particularly required for clayey and bouldery strata. Soundings, wherever possible,shall be taken in the vicinity of the site for the proposed bridge and for any structures nearby. Such soundings are best during or immediately after a flood before the scour holes have had time to be silted up. The mean scour depth may be fixed based on such observations and theoretical calculation.

703.3 Maximum Depth of Scour for Design of Foundation

703.3.1 The maximum depth of scour below the Highest Flood Level (HFL) for the design of piers and abutments having individual foundations without any floor protection may be considered as follows.

703.3.1.1 *Flood without seismic combination*

i) For piers - $2.0\ d_{sm}$

ii) For abutments - a) $1.27\ d_{sm}$ with approach retained or lowest bed level whichever is deeper.

b) $2.00\ d_{sm}$ with scour all around.

703.3.1.2 *Flood with seismic combination*

For considering load combination of flood and seismic loads (together with other appropriate combinations given elsewhere) the maximum depth of scour given in **Clause 703.3.1.1** may be reduced by multiplying factor of 0.9.

703.3.1.3 For low water level (without flood conditions) combined with seismic combination maximum level of scour below high flood level can be assumed as 0.8 times scour given in **Clause 703.3.1.**

Note: In respect of viaduct/ROB having no possibility of scour, resistance of soil may be considered below depth of excavation for services construction, or 2.0 m below ground level whichever is greater.

703.3.2 For the design of floor protection works for raft or open foundations, the following values of maximum scour depth may be adopted:

i) In a straight reach $1.27\ d_{sm}$

ii) In a bend $1.50\ d_{sm}$ or on the basis of concentration of flow

The length of apron on upstream may be 0.7 times of the same on downstream.

703.4 Special studies should be undertaken for determining the maximum scour depth for the design of foundations in all situations where abnormal conditions, such as,the following are encountered:

i) A bridge being located on a bend of the river involving a curvilinear flow, or excessive shoal formation, or

ii) A bridge being located at a site where the deep channel in the river hugs to one side, or

iii) A bridge having very thick piers inducing heavy local scours, or

iv) Where the obliquity of flow in the river is considerable, or

v) Where a bridge is required to be constructed across a canal, or across a river downstream of storage works, with the possibility of the relatively clear water inducing greater scours, or

vi) A bridge in the vicinity of a dam, weir, barrage or other irrigation structures. Where concentration of flow, aggradations/degradation of bed, etc. are likely to affect the behaviour of the structures, or

vii) An additional two-Lane Bridge when located near to the existing bridge, on major rivers.

Note: These studies shall be conducted for the increase discharge calculated vide **Clause 703.1.1**

703.5 If a river is of flashy nature and bed des not lend itself readily to the scouring effect of floods, the theoretical formula for d_{sm} and maximum depth of scour as recommended shall not apply. In such cases, the maximum depth shall be assessed from actual observations.

704 SUB-SURFACE EXPLORATION

704.1 Objectives

The objectives of the sub-surface exploration are:

i) During Preliminary Investigation Stage

As a part of the site selection process to study existing geological maps and other information, previously prepared and available site investigation reports, known data of nearby structures, if any, surface examination of river beds. banks, etc., which will help in narrowing down of sites under consideration for further studies for the project preparation stage.

ii) Detailed Investigation Stage

To determine the characteristics of the existing geomaterials, like, soil, rock, bed material in watercourses, etc., in the zone of influence of the proposed bridge sites in such a way as to establish the design parameters which influence the choice and design details of the various structural elements, especially the foundation type.

iii) During Construction Stage

To confirm the characteristics of geomaterials established in stage (ii) based on which the design choices are made and to re-confirm the same or modify to suit the conditions met at specific foundation locations.

704.2 Zone of Influence

The zone of influence mentioned in Clause 704.1 (ii) is defined as the full length of the bridge including the portion of the wing/return wall and part of approaches covering, (but not restricted to), the full flood zone for watercourses, and up to a depth below the proposed foundation levels where the influence of stresses due to the loads on the foundation is likely to affect the behaviour of the structure, including settlement, subsidence underground flow of water, etc. The

ıth of the land strip on either side of the proposed structure should include a zone in which the ydraulic characteristics of river water are likely to be changed affecting flow patterns, scour, etc.

704.3 Methods of Exploration

A large variety of investigative methods are available. An ost suitable and appropriate combination of these shall be chosen. Guidelines for choice of types of investigations, properties of geo-materials that need to be established, the in-situ testing, sampling, and laboratory testing are given in **Appendix-1**. This may be further supplemented by specialized techniques depending on the need.

704.4 Geophysical Investigations

For geophysical investigation, reference may be made to IRC:123.

705 DEPTH OF FOUNDATION

705.1 General

The foundation shall be designed to withstand the worst combination of loads and forces evaluated in accordance with the provisions of Clause 706. The foundations shall be taken to such depth that they are safe against scour or protected from it. Apart from this, the depth should also be sufficient from consideration of bearing capacity, settlement, liquefaction potential, stability and suitability of strata at the founding level and sufficient depth below it.

When a new bridge is proposed near an existing bridge, the minimum clear gap between the new and old foundations should be such that it does not cause any adverse effect on the existing structure even during the construction stage, such as sand blow condition.

In the case of foundations on soil, if a new foundation is proposed near an existing foundation, the safety/stability of the existing foundation shall be checked during construction as well as in the service stage. It shall be ensured that i) the bearing pressures under both foundations are within permissible limits, and ii) the existing foundation is safe under sand blow conditions if it arises during construction of new bridge.

705.2 Open Foundation

705.2.1 *In soil*

The embedment of foundations in soil shall be based on a correct assessment of anticipated scour.

Foundation may be taken down to a comparatively shallow depth below the bed surface provided a good bearing stratum is available, and the foundation is protected against scour.

It shall be ensured that the foundation rests on firm strata having safe bearing capacity equal to or greater than maximum design base pressure. All loose material below the foundation shall be removed and replaced by granular material.

The minimum depth of various types of open foundations shall be as below:

i) In the case of an isolated footing or raft foundation without floor protection, the minimum depth of foundation shall be 2m below the maximum scour level.

ii) In the case of an isolated foundation with floor protection, the minimum depth of the foundation shall be 2m below the top of the floor level.

iii) In the case of raft foundation with floor protection, the top of the raft shall be kept at a minimum of 300 mm depth below the deepest bed level in the river cross-section on the bridge alignment.

iv) In the case of retaining walls on land (away from waterbody), minimum depth of foundation below finished/natural ground level shall be 1.5m.

v) The depth of foundation for short height retaining walls (less than 1.5m above Ground level) shall not be less than 1 m

vi) For Reinforced Soil Walls Reference may be made to IRC:SP-102 regarding depth of foundation.

Note: *The protection works shall be provided as per IRC:89.*

705.2.2 *In rocks*

For open foundations resting on rock, the depth of rock, which in the opinion of the geological expert is weathered or fissured, shall be excluded in deciding the depth of embedment into the rock existing below. Where foundations are to rest on credible rocks, caution shall be exercised to establish the foundation level at sufficient depth, so as to ensure that they do not get undermined, keeping in view the continued erosion of the bed. After allowing for conditions stipulated above the minimum embedment of the foundations into the rock shall be as per Table given below:

	Type of Rock	**Embedment depth**
a)	For rocks of moderately strong and above in Table 2 of classification of rock (under Clause 8.6 of **Appendix-1**) having UCS of more than 12.5 MPa or where it is not possible to take core to get the UCS but extrapolated SPT N value is more than 500	0.6 m
b)	For rock of moderately weak and below in Table 2 of classification of rock (under Clause 8.6 of **Appendix-1**) having UCS< 12.5 MPa but extrapolated SPT N value is more than 100 but less than 500	1.5 m

In the case of sloping rock profile, proper benching shall be provided in the rock before laying the foundation.

705.3 Well Foundations

705.3.1 *In soil*

Well foundations shall be taken down to a depth which will provide a minimum grip of 1/3rd of the maximum depth of scour below the maximum scour level.

705.3.2 *In rocks*

As far as possible, the wells shall be taken by all the methods of sinking including pneumatic sinking (where considered necessary), dewatering, etc. to foundation level and shall be evenly seated all around the periphery on sound rock (i.e., devoid of fissures, cavities, weathered zone, likely extent of erosion, etc.) by providing adequate embedment. The extent of seating and embedment in each case shall be decided by the Engineer-in-charge keeping in view the factors mentioned above to ensure overall and long-term safety of the structure. A shear key of 300 mm

in hard rock or 600 mm in soft rock shall be made inside the well by chiselling/blasting. Diameter of shear key shall be 1.5 to 2 m less than inner dredge-hole subject to a minimum size of 1.5 m.

705.4 Pile Foundations

705.4.1 In soil, the minimum depth of foundations below the point of fixity should be the minimum length required for developing full fixity as calculated by any rational formula.

705.4.2 In rocks, the pile should be taken down to rock strata devoid of any likely extension of erosion and properly socketed as required by the design.

706 LOADS, FORCES, STABILITY AND STRESSES

706.1 Loads, Forces and their Combinations

Table 706.1: Load Combinations for checking of Foundation Pressure (Clause 706.1.1)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	G	Q	G_s	F_{im}	V_c	W	F_{wc}	(F_a or f_b) & F_f	F_{cf}	G_b	F_{ep}	F_{te}	F_d	F_s	F_{er}	F_{eq}	F_{wp}	F_p	

	Dead load (G)	Live Load (Q)	Snow Load (G_s)	Impact of Floating bodies (F_{im})	Vehicle Collision Load (V_c)	Wind (W)	Water Current (F_{wc})	Tractive (F_a)	Braking (F_b)	Bearing Friction(F_f)	Centrifugal Force (F_{cf})	Buoyancy (G_b)	Earth pressure (F_{ep})	Temperature (F_{te})	Deformation Effects (F_d)	Seconuary Effects (F_s)	Erection Effects (F_{er})	Seismic (F_{eq})	Wave Pressure (F_{wp})	Prestress Primary & Secondary effects (F_p)	Remarks
I	1	1	*				1	1	1	1	1	1	1						1	1		Service Condition
II	1	1	*				1	1	1	1	1	1	1		1	1			1	1		
III	1	1	*			1	1	1	1	1	1	1	1	1					1	1		
IV	1	1	*			1	1	1	1	1	1	1	1						1	1		
V	1				1															1		
VI	1	0.2					1	0.2	0.2	1	0.2	1	1	1				1	1	1		
VII	1	1	*	1		1	1	1	1	1	1	1	1	1					1	1		
VIII	1					1	1			1		1	1				1			1		Construction Condition
IX	1						1			1		1	1				1	0.5		1		

706.1.1 The loads and forces may be evaluated as per IRC:6 and the foundation pressure shall be checked for the load combinations shown in **Table 706.1**.

706.1.2 The permissible increase in allowable base pressure shall be 25 percent for all combinations except combination I. However, when temperature effects (F_{te}), secondary effects (F_s) deformation effects (f_d) are also to be considered for any members in combination with I then permissible increase in allowable bearing pressure will be 15 percent.

For all combinations with SV loading as live load, the permissible increase in allowable base pressure shall be 25%.

Alternative to the above design approach, limit state design approach can be followed for which IRC:78 Part 2 may be referred to.

706.1.3 For simply supported spans, substructure and foundation shall also be checked for 'One span Dislodged' condition. This shall be treated as Combination VIII. On span in position, provision for a live load of 100 kg/sqm shall also be considered to account for additional loading during restoration of the dislodged span.

706.2 Horizontal Forces at Bearing Level

Horizontal forces generated by the bearings or resisted by bearings both in case of simply supported and continuous spans shall be calculated in accordance with IRC: 6.

706.2.1 *Simply supported spans*

Deleted. Refer IRC:6

706.2.2 *Simply supported and continuous span on flexible supports*

Deleted. Refer IRC:6

706.3 Base Pressure

706.3.1 The allowable bearing pressure and the settlement characteristics under different loads and stresses may be determined on the basis of sub-soil exploration and testing. Though the help of relevant Indian Standard Code of Practice may be taken, the allowable bearing pressure may be calculated as gross so that the gross pressure at the base without deducting the soil displaced, the soil displace can be computed.

706.3.1.1 For open foundations and well foundation resting on soil, the allowable bearing pressure on ultimate bearing capacity may be taken as 2.5 for soil.

706.3.1.1.2 For open foundations and well foundation resting on rock, the allowable bearing pressure on rock may be decided upon not only on the strength of parent rock but also on overall characteristics particularly deficiencies, like, joints, bedding planes, faults, weathered zones, etc. In absence of such details or analysis of overall characteristics, the value of factor of safety based on unconfined compressive strength of the parent rock may be taken as 6 to 8 unless otherwise indicated on the basis of local experience. The allowable bearing pressure, thus, obtained is to be further restricted to not over 3 MPa for load combination I. For Factor of safety in case of pile foundation the Clause 709.3.2 shall be referred to.

The intermediate geo-material like disintegrated weathered or very soft rock may be treated as soil.

706.3.2 *Allowable total and differential settlement*

706.3.2.1 The calculated differential settlement between the foundation of simply supported spans shall not exceed 1 in 400 of the distance between the two foundations form the consideration of tolerable riding quality unless provision has been made for rectification of this settlement.

In case of structures sensitive to differential settlement, the tolerable limit has to be fixed for each case separately.

706.3.2.2

For foundations resting on soil, the maximum allowable total settlement due to loads from superstructure, shall be as below:

For open foundations – 75 mm

For Well foundation – 150 mm

The foundation and substructure shall be designed for a minimum differential settlement as mentioned below:

For open foundations resting of soil – 15 mm

For Well foundation resting on soil – 25 mm

Pile foundation resting on soil – 1% of pile diameter

For foundations resting on rock – NIL

706.3.3 *Permissible loss of contact at the base of foundation*

706.3.3.1 For Foundation resting on soil ,full contact of the base shall be ensured under all combination of loads.

706.3.3.2 In case of rock if loss of contact is found to be developed at the base of foundation, the base area should be reduced to a size where no tension will occur and base pressure is recalculated. The maximum pressure on such reduced area should not exceed allowable bearing pressure. Such reduced area shall not be less than 67 percent of the total area for load combination including F_{eq} (seismic), or F_{im} (impact of barge), and 80 percent for other load combinations.

The piles resting on soil or rock may be subjected to uplift under any combination of loads.

706.3.4 *Factor of safety for stability*

Factors of safety against overturning and sliding are given below. These are mainly relevant for open foundation:

	Without Seismic Case	**With Seismic and SV loading Case**
i) Against overturning	2	1.50
ii) Against sliding	1.5	1.25
iii) Against deep-seated failure	1.25	1.15

Frictional co-efficient between concrete and soil/rock will be Tan ø, ø being angle of friction. Founding soil in foundation of bridge being generally properly consolidated, following values may be adopted:

Friction co-efficient between soil and concrete = 0.5

Friction co-efficient between rock and concrete = 0.8 for good rock and 0.7 for fissured rock.

706.3.5 *Deleted*

707 OPEN FOUNDATIONS

707.1 General

707.1.1 Provision of this Clause shall apply for design of isolated footings and, where applicable, to combined footings, and rafts.

707.1.2 Open foundations may be provided where the foundations can be laid in a stratum which is inerodible or where the extent of scour of the bed is reliably known. Wherever required, the foundations shall be reliably protected by means of suitably designed apron, cut-off wall, and/or launching apron in accordance with IRC:89.

707.1.3 To ensure proper load transfer, the ratio of depth to length/width of footing shall not exceed 1:3.

707.2 Design

707.2.1 The thickness of the footings shall not be less than 300 mm.

707.2.2 to 707.2.8 Deleted, refer IRC:112

707.2.9 In case of footing in plain cement concrete, brick masonry or stone masonry, the load from the pier or column shall be taken as dispersed through depth of footing at an angle not exceeding 45° to vertical.

707.3 Open Foundations at Sloped Bed Profile

707.3.1 Open foundations may rest on sloped bed profile provided the stability of the slope is ensured. The footings shall be laid on a horizontal base.

707.3.2 For the adjacent foundations, the effect of dispersion of pressure from the foundation laid on the higher level should be duly considered on the foundation at the lower level. The distance between two foundations at different levels may be decided in such a way to minimize this effect taking into account the nature of soil.

The distance between the two foundations at different levels may be decided in such a way to minimize this effect taking into account the nature of soil.

707.4 Construction

707.4.1 The protective works shall be completed before the floods so that the foundation does not get undermined.

707.4.2 Excavation on open foundations shall be done after taking necessary safety precautions for which guidance may be taken from IS:3764.

707.4.3 Where blasting is required for excavation in rock, and is likely to endanger adjoin foundations or other structures, necessary precautions such as, controlled blasting, providing suitable mat cover to prevent flying of debris etc. shall be undertaken to prevent any damage to these structures.

707.4.4 *Condition for laying of foundations*

707.4.4.1 Normally, the open foundations should be laid dry and every available method of dewatering by pumping or depression of water by well point, etc. may be resorted to. A levelling course of 100 mm thickness in M 15 grade concrete shall be provided below foundation.

707.4.4.2 If it is determined before-hand the foundations cannot be laid dry or the situation is such that the percolation is too heavy for keeping the foundation dry, the foundation concrete may be laid under water only by tremie pipe. In case of flowing water or artesian springs, the flow shall be stopped or reduced as far as possible at the time of placing of concrete. No pumping of water shall be permitted form the time of placing of concrete upto 24 hours after placement.

707.4.5 All spaces excavated and not occupied by abutments, pier or other permanent works shall be refilled with earth upto the surface of the surrounding ground, with sufficient allowance for settlement. All backfill shall be thoroughly compacted and in general, its top surface shall be neatly graded.

707.4.6 In case of excavation in rock, the trenches around the footing shall be filled-up with concrete of M 15 grade upto top of the rock.

707.4.6.1 If the depth of fill required is more than 1.5 m in soft rock or 0.6 m in hard rock above the foundation level, then concrete may be filled upto this level by M 15 concrete and portion above may be filled by concrete or by boulders grouted with cement-sand grout.

707.4.6.2 For design of foundation on rock in river bridges, the design loads and forces shall be considered upto the bottom of footing. The load of filling need not be considered in stability calculations.

707.5 Raft Foundations

Guidelines for Raft foundations given in **Appendix-2** may be adopted wherever applicable.

708 WELL FOUNDATION

708.1 General

708.1.1 Foundations supporting the superstructure located in deep water Channels shall comprise of properly dimensioned wells. While selecting the shape, size and type of well, the size of abutment and pier to accommodate, need for effecting streamline flow, the possibility of the use of pneumatic sinking, the anticipated depth of foundation and the nature of data to be penetrated should be kept in view. The minimum dimensions of dredge-hole shall not be less than 3 m. In case there is deep standing water, properly designed floating caissons may be used as per Clause 708.12.

However, in the case of a larger bridges across rivers in wide flood plains prone to scour, deltaic tidal rivers, channels with inland waterway traffic and bridges in coastal/ marine locations, the number of intermediate foundations shall be reduced as far as practicable.

708.1.2 If the external diameter of a single circular well exceeds 12m, then Engineer- in Charge shall take recourse to any of the following:

a) Evaluate Stress in steining using the 3-Dimensional Finite Element Method (3D FEM) or any other suitable analytical method.

b) Stiffening by compartments. The design of such stiffened wells shall call for supplemental design and construction specification,

708.1.3 The conditions arising out of sand blow, if anticipated, should be duly considered.

708.1.4 Bottom plug of well should be suitably designed to resist maximum upward force acting on it during construction following plugging as well as during life span of the structure.

708.2 Well Steining

708.2.1 Thickness of the steining should be such so that it is possible to sink the well without excessive kentledge and without getting damaged during sinking or during rectifying the excessive tilts and shifts. The steining should also be able to resist differential earth pressure developed during sand blow or other conditions, like, sudden drop.

Stresses at various levels of the steining should be within permissible limits under all conditions for loads that may be transferred to the well.

708.2.2 Use of cellular steining with two or more shells or use of composite material in well steining shall not be permitted for wells upto 12m diameter.

708.2.3 *Steining thickness*

708.2.3.1 The minimum thickness of the well steining shall not be less than 500 mm and satisfy the following relationship:

$h = Kd\sqrt{l}$

h = minimum thickness of steining in metre

d = external diameter of circular well of dumb bell shaped well or in case of twin D wells smaller dimension in plan in metres

l = depth of wells in metre below top of well cap or LWL whichever is more (for floating caisson 'l' may-be taken as depth of well in metres below bed level.)

K = a constant

Value of K shall be as follows:

i) Well in cement concrete K = 0.03

ii) Well in brick masonry K = 0.05

iii) Twin D wells K = 0.039

708.2.3.2 The minimum steining thickness may be varied from above in following conditions:

Strata	Variation from the minimum	Recommended variation upto
a) Very soft clay strata	Reduced	10%
b) Hard clay strata	Increased	10%
c) Bounder strata or well resting on rock involving blasting	Increased	10%

708.2.3.3 However, following aspects may also be considered depending on the strata:

a) **Very soft clay strata:** Main criteria for reduction in steining thickness in to prevent the well penetrating by its own weight. When the thickness is so reduced, the steining shall be adequately reinforced, to get sufficient strength.

b) **Hard clay strata:** Depending on the previous experience, the increase in steining thickness may be more than 10 percent.

c) **Bouldery strata or well resting on rock involving blasting:** higher grade of concrete, higher reinforcement, use of steel plates in the lower portions, etc., may be adopted.

708.2.3.4 The recommended values given in Clauses 708.2.3.2 can be further varied based on local experience and in accordance with decision of Engineer-in-Charge.

708.2.3.5 If specialised methods of sinking, such as, jack down method are adopted then the steining thickness may be adjusted according to design and construction requirements.

708.2.3.6 Deleted.

708.2.3.7 When the depth of well below well cap is equal to or more than 30 m, the thickness of the steining of the well calculated as per Clause 708.2.3 may be reduced above scour level in a slope of 1 horizontal to 3 vertical such that the reduced thickness of the steining should not be less than required as per Clause 708.2.3 for the depth of well upto scour level with the reduced diameter.

The reduction in thickness shall be done in the outer surface of the well. The diameter of inner dredge hole shall be kept uniform.

The minimum steel and the concrete grade in the slope portion shall be same as for the steining below scour level.

Minimum development length of all the vertical steel bars shall be provided beyond the minimum section as shown in the **Appendix-3 (Fig. 1)**.

The stress in the reduced section of steining shall also be checked.

708.3 Design Considerations

708.3.1 The external diameter of the brick masonry wells shall not exceed 6 m. Brick masonry wells for depth greater than 20 m shall not be permitted.

708.3.2 For brick masonry wells, brick not less than Grade–A having strength not less than

7 MPa – conforming to IS:1077 shall be used in cement mortar not leaner than 1:3.

708.3.3 For plain concrete wells, vertical reinforcements (whether mild steel or deformed bars) in the steining shall not be less than 0.12 percent of gross sectional area of the actual thickness provided. This shall be equally distributed on both faces of the steining . The vertical reinforcements shall be tied up with hoop steel not less than 0.04 percent of the volume per unit length of the steining, as shown in the **Appendix-3 (Fig.2)**.

708.3.4 In case where the well steining is designed as a reinforced concrete element, it shall be considered as a column section subjected to combined axial load and bending. However, the amount of vertical reinforcement provided in the steining shall not be less than 0.2 percent (for either mild steel or deformed bars) of the actual gross sectional area of the steining. On the inner face, a minimum of 0.06 percent (of gross area) steel shall be provided. The transverse reinforcement in the steining shall be provided in accordance with the provisions for column but in no case shall be less than 0.04 percent of the volume per unit length of the steining.

The horizontal annular section of well steining shall also be checked for ovalisation moments by any rational method taking account of side earth pressures evaluated as per Clause 708.4.

708.3.5 The vertical bond rods in brick masonry steining shall not be less than 0.1 percent of the cross- section area and shall be encased into cement concrete of M 15 mix of size 150 mm x 150 mm. These rods shall be equally distributed along the circumference in the middle of the steinging and shall tied up with hoop steel not less than 0.04 percent of the volume per unit length of the steining. The hoop steel shall be provided in a concrete band at spacing of 4 times of the thickness of the steining or 3 metres, whichever is less. The horizontal RCC bands shall not be less than 300 mm wide and 150 mm high, reinforced with bars of diameter not less than 10 mm placed at the corners and tied with 6 mm diameter stirrups at 300 mm centres, as shown in the **Appendix-3 (Fig. 3)**.

708.3.6 The stresses in well steining shall be checked at such critical sections where tensile and compressive stresses are likely to be maximum and also where there is change in the area of reinforcement or in the concrete mix.

708.4 Stability of Well Foundations

708.4.1 The stability and design of well foundations shall be done under the most critical combination of loads and forces as per Clause 706. The pressure on foundations shall satisfy the provisions of Clause 706.

708.4.2 *Side earth resistance*

708.4.2.1 The side earth resistance shall be calculated as per guidelines given in **Appendix-3**.

708.4.2.2 The side earth resistance shall be ignored in case of well foundations resting on rock. If rock strata is such that the allowable bearing pressure is less than 1 MPa, then the side earth resistance may be taken into account.

708.4.3 *Earth pressure on abutments*

708.4.3.1 If the abutments are designed to retain earth and not spilling in front, the foundations of such abutments shall be designed to withstand the earth pressure and horizontal forces for

the condition of scour depth in front of $1.27d_{sm}$ with approach retained and $2d_{sm}$ with scour all around. In case of scour all around, live load may not considered.

708.4.3.2 However, where earth spilling from the approaches is reliably protected in front, passive relief due to the spilling earth in front may be considered from bottom of well cap downwards.

708.4.4 *Construction stage*

708.4.4.1 Stability of the well shall also be checked for the construction stage when there is no superstructure and the well is subjected to design scour, full pressure due to water current and/ or full design earth pressure as in the case of abutment wells.

708.4.4.2 During the construction of wells when it has not reached the founding level or has not been plugged, the wells are likely to be subjected to full pressure due to water current upto full scour. This may result in tilting, sliding and shifting. As a part of the safety during construction, this should be considered and safety of well must be ensured by suitable methods, where required.

708.5 Tilts and Shifts

708.5.1 As far as possible, the wells shall be sunk plumb without any tilts and shifts. However, a tilt of 1 in 80 and a shift of 150 mm due to translation (both additive) in a direction which will cause most severe effect shall be considered in the design of well foundations.

708.5.2 If the actual tilts and shifts exceed the above limits, then the remedial measures have to be resorted to bring the well within than limit. If it is not possible then its effect on bearing pressure, steining stress and other structural elements shall be examined, and controlled if necessary and feasible, by resorting to change in span length.

708.6 Cutting Edge

708.6.1 The mild steel cutting edge shall be strong enough and not less than 40 kg/m to facilitate sinking of the well through the types of strata expected to be encountered without suffering any damage. It shall be properly anchored to the well curb. For sinking through rock, cutting edge should be suitably designed.

708.6.2 When there are two or more compartments in a well, the lover end of the cutting edge of the middle stems of such wells shall be kept about 300 mm above that of the outer stems to prevent rocking, as shown in the **Appendix-3 (Fig. 2)**.

708.7 Well Curb

708.7.1 The well curb should be such that it will offer the minimum resistance while the well is being sunk but should be strong enough to be able to transmit superimposed loads from the steining to the bottom plug.

708.7.2 The shape and the outline dimension of the curb as given in **Appendix-3 (Fig. 2)** maybe taken for guidance. The internal angle of the curb 'as' as showr in **Appendix-3 (Fig. 2)** should be kept at about 30° to 37° and may be increased or decreased based on past experience and geotechnical data.

708.7.3 The well curb shall invariably be in reinforced concrete of mix not leaner than M 25 with minimum reinforcement of 72 kg/cum excluding bond rods. The steel shall be suitably arranged to prevent spreading and splitting of the curb during sinking and in service.

708.7.4 In case blasting is anticipated, the inner faces of the well curb shall be protected with steel plates of thickness not less than 10 mm upto the top of the well curb. If it is desired to increase the steel lining above the well curb then the thickness can be reduced to 6 mm for that increased height. In any case, this extra height of the steel should not be more than 3 metres unless there is a specific requirement. The curb in such a case should be provided with additional hoop reinforcement of 10 mm dia mild steel or deformed bars at 150 mm centres which shall also extend upto a height of 3 m into the well steining above the curb. Additional reinforcement above this height upto two times the thickness of steining should be provided to avoid cracking arising out of sudden change in the effective section due to curtailment of plate.

708.8 Bottom Plug

708.8.1 The bottom plug shall be provided in all wells and the top shall be kept not lower than 300 mm in the centre above the top of the curb as shown in the **Appendix-3 (Fig. 2)**. A suitable sump shall be below the level of the cutting edge. Before concerting the bottom plug, it shall be ensured that its inside faces have been cleaned thoroughly.

708.8.2 The concrete mix used in bottom plug shall have a minimum cement content of 330 kg/m^3 and a slump of 150 mm to permit easy flow of concrete through tremie to place fill-up all cavities. Concrete shall be laid in one continuous operation till dredge hole is filled to required height. For under water concreting, the concrete shall be placed by tremie under still water condition and the cement content of mix be increased by 10 percent.

708.8.3 In case grouted concrete, e.g. concrete is used, the grout mix shall not be leaner than 1:2 and it shall be ensured by suitable means, such as, controlling the rate of pumping that the grout fills-up all interstices upto the top of the plug.

708.8.4 If any dewatering is required it shall be carried out after 7 days have elapsed after bottom plugging.

708.9 Filling the Well

708.9.1 The filling of the well, if considered necessary, above the bottom plug shall be done with sand or excavated material free from organic matter.

708.10 Plug over Filling

708.10.1 A 300 mm thick plug of minimum M 15 cement concrete shall be provided over the filling.

708.11 Well Cap

708.11.1 The bottom of well cap shall be laid as low as possible but above the LWL in the active channel. Where the bed level is higher than LWL the bottom of well cap may be suitably raised.

708.11.2 As many longitudinal bars as possible coming from the well steining shall be anchored into the well cap.

708.11.3 The design of the well cap shall be based on; any accepted rational method, considering the worst combination of loads and forces as per **Clause 706/Table 706.1**

708.12 Floating caissons

708.12.1 Floating caissons may be of steel, reinforced concrete or any suitable material. They should have at least 1.5 metres free board above the water level and increased, if considered necessary, in case there is a possibility of caissons sinking suddenly owing to reasons, such as, scour likely to result from lowering of caissons, effect of waves, sinking in very soft strata, etc.

708.12.2 Well caissons should be checked for stability against overturning and capsizing while being towed, and during sinking, due to the action of water current, wave pressure, wind, etc.

708.12.3 The floating caisson shall not be considered as part of foundation unless proper shear transfer at the interface is ensured.

708.13 Sinking of Wells

708.13.1 The well shall as far as possible be sunk true and vertical. Sinking should not be started till the steining has been cured for at least 48 hours. A complete record of sinking operating including tilt and shifts, kentledge, dewatering, blasting, etc. done during sinking shall be maintained.

For safe sinking of wells, necessary guidance may be taken from the precautions as given in **Appendix-4**.

708.14 Pneumatic Sinking of Wells

708.14.1 Where sub-surface data indicate the need for pneumatic sinking, it will be necessary to decide the method and location of pneumatic equipment and its supporting adapter.

708.14.2 In case if concrete steining is provided, it shall be rendered air tight by restricting the tension in concrete which will not exceed 318^{th} of the modulus of rupture. For the circular wells, the tension in steining may be evaluated by assuming it to be a thick walled cylinder.

708.14.3 The steining shall be checked at different sections for any possible rupture against the uplift force and, if necessary, shall be adequately strengthened.

708.14.4 The design requirements of the pneumatic equipment, safety of personnel and the structure shall comply with the provisions of IS:4138 "Safety Code for Working in Compressed Air". It is desirable that the height of the working chamber in a pneumatic caissons should not be less than 3 metres to provide sufficient head room when the cutting edge is embedded a short distance below the excavated level and in particular to allow for blowing down. The limiting depth for pneumatic sinking should be such that the depth of water below normal water level to the proposed foundation level upto which pneumatic sinking is proposed should not exceed 30m.

708.15 Sinking of Wells by Resorting to Blasting

Blasting may be employed with, prior approval of competent authority to help sinking of well for breaking obstacles, such as, boulders or for levelling the rock layer for square seating of wells. Blasting may be resorted to only when other methods are found ineffective.

709 PILE FOUNDATION

709.1 General

709.1.1 Piles transmit the load of a structure to competent sub-surface strata by the resistance developed from end bearing at the toe or skin friction along the surface or both. The piles may be required to carry uplift and lateral loads besides direct vertical load.

709.1.2 The construction of pile foundation requires a careful choice of piling system depending upon subsoil conditions and load characteristics of structures. The permissible limits of total and differential settlement, unsupported length of pile under scour and any other special requirements of project are also equally important criteria for adoption.

709.1.3 Design and construction: The geotechnical design and construction of piles shall be carried out as per this standard. **Appendix-5** gives the design formulae and their applicability.

709.1.4 For piles in streams, rivers, creeks, etc. the following criteria may be followed:

i) Scour conditions are properly established.

ii) Permanent steel liner should be provided at least upto maximum scour level. In case of marine clay or soft soil or soil having aggressive material, permanent steel liner of sufficient strength shall be used for the full depth of such strata. The minimum thickness of liner should be 6 mm

For bridges located in land, steel liners of minimum thickness of 6 mm shall be provided in cases given below. The liner shall be provided up to depth up to which following situations prevail.

a) While constructing the pile foundation through very soft clay (N <3), very loose sandy strata (N < 8), bouldery formation and artesian conditions, wherein the walls of boreholes cannot be stabilized by bentonite circulation.

b) Where sewage leakage is common phenomenon as well as sites with aggressive soil/ water environment

709.1.5 Spacing of piles and tolerances

709.1.5.1 Spacing of piles

a) **Multiple Piles:** Where pier is supported on multiple piles, connected by frame structure or by solid pile cap, the spacing of piles should be considered in relation to the nature of the ground, their behaviour in groups and the execution convenience. The spacing should be chosen with regard to the resulting heave or compaction and should be wide enough to enable the desired number of piles to be installed to the correct penetration without damage to any adjacent construction or to the piles themselves.

b) **Mono Piles:** Pier may be supported on mono pile having minimum diameter of 2.5m.

The pile should be designed to cater for the maximum eccentricity of vertical load considering construction tolerances.

Alternatively, pile shaft can be continued to act as a pier and get connected to pier cap which is designed to accommodate the eccentricities due to construction tolerances.

The size of a cap carrying the load from the structure to the pile heads, or size and effective length of a ground beam; may influence type: size and spacing of piles.

The spacing of piles will be determined by many aspects mentioned above. The working rules which are generally, though not always, suitable, are as follows:

For friction piles, the centre to centre spacing should not be less than the perimeter of the pile or for circular pile, three times the diameter. The spacing of piles deriving their resistance mainly from end bearing may be reduced but the distance between the surfaces of the shafts of adjacent piles should be not less than the least width/diameter of the piles.

709.1.5.2 Setting out and permissible tolerances for piles shall be as under:

i) Setting out shall be carried out as per the design pile coordinates to locate the positions of piles.

ii) The pile head position of vertical piles can have a maximum deviation of 75mm at piling platform level.

iii) For Vertical piles the tilt measured shall not exceed 1 in 150.

iv) For raker piles tolerance of 1 in 25 from the design axis.

709.1.6 The maximum rake to be permitted in pile shall not exceed the following:

i) 1 in 6 for all bored piles;

ii) 1 in 6 for driven cast-in-situ piles; and

iii) 1 in 4 for precast driven piles.

709.1.7 The minimum diameter shall be 1.0 m for river/marine bridges. For bridges beyond the water zone and for bridges on land, the diameter may be reduced upto 750 mm.

709.1.8 *Settlement, differential settlement and pile capacity*

The Differential settlement between two successive foundations taken at pile cap level, may be estimated from the maximum settlement expected at two foundations for the dead load, superimposed loads, live load and scour effect. The increase in settlement with time in clayey soils shall be accounted for. In absence of detailed calculations, for the purpose of preliminary design, it can be taken as not more than the maximum settlement of any of the two foundations.

The differential settlement shall be limited depending upon the following functional and structural considerations:

a) Functionally, acceptable differential settlement between two neighboring piers shall not be greater than 1 in 400 of the span to ensure riding comfort, as specified in Clause 706.3.2.1.

b) The allowable settlement of a single pile considered for estimating the pile capacity shall be arrived from correlation of the settlement of pile grcup to that of single pile, as per Clause 709.3.4.

c) Based on the allowable settlement of single pile, the working load capacity of pile shall have a factor of safety of 2.5 on ultimate load capacity. The working load capacity shall not exceed 40 percent of ultimate load corresponding to a settlement of 10 percent of pile diameter.

709.1.9 For both Precast and cast-in-situ piles, the values regarding grade of concrete, water cement ratio, slump shall be as follows:

	Concrete Cast-in-Situ by Tremie	Precast Concrete
Min. Grade of concrete	M 35	M 35
Min. cement contents	400 kg/m^3	400 kg/m^3
Max. W.C. ratio	0.4	0.4
Slump (mm) - range	150-200	50-75

Note: i) For improving resistance to penetration of harmful elements from soil use of mineral admixtures (flyash, silica fume, GGBS conforming to respective BIS/International Standards) and as per IRC:112 is recommended.

ii) The Minimum grade of concrete should be M40 for class 5 sulphate content of Table 14.4 of IRC:112

709.2 Requirement and Steps for Design and Installation

709.2.1 *Initial vertical load test:*

The initial design of an individual pile, confirmation of its capacity by either initial load test or by re-confirmation of actual soil parameters, modification of design, if required, and final adoption should pass through following steps of investigations, design and load testing:

i) Comprehensive and detailed sub-surface investigation for piles to determine the design parameters of end bearing capacity, friction capacity and lateral capacity of soil surrounding the pile.

ii) Design of pile and pile group based on i) above for specified bearing strata.

iii) Initial load testing:

Initial load test on pile of same diameter, length and capacity as envisaged in design.

The initial load test is a part of the design process confirming the expected properties of bearing strata and the pile capacity.

iv) Steps ii) and iii) should be repeated for different types of strata met at site.

709.2.2 Steps for design and confirmation by tests are given below:

i) Sub-soil exploration to reconfirm soil parameters assumed in the design.

ii) Provide for the required design capacity of pile group based on tentative number and diameter of piles in a group.

iii) The allowable total/differential settlement of single pile should be based or the considerations as per Clauses 709.1.8 and 709.3.4. Capacity of single pile is to be based on static formula considering ground characteristics. This step along with step ii) may be iterative.

iv) Structural design of piles.

v) Initial load test as mentioned in Clause 709.2.1 iii) is for axial load capacity, including uplift capacity, if required, on trial piles of the same diameter as the design pile. The testing shall be done as per the procedure laid down in IS: 2911, Part-IV. Alternatively, the test procedure provided for Bi – directional pile load testing as given in **Appendix-7** may be adopted. This load test shall be conducted for not less than 2.5 times the design load. The initial load test shall be cyclic load test for piles deriving trength from end bearing and side friction. The maintained load test can be performed for end bearing piles without relying on friction, and for the socketed piles in rock.

vi) If the initial load test gives a capacity greater than 25 percent of the capacity calculated by static formula, and if it is desired to take benefit of the higher capacity, another two load tests shall be carried out to confirm the earlier value and minimum of the three & shall be considered as initial load test value. The number of initial tests shall be determined by the Engineer-in-Charge taking into consideration the bore log and soil profile.

709.2.3 For abutment, it is important to consider overall stability of the structure and abutment. The piles should also be designed to sustain surcharge effect of embankment.

709.2.4 *Routine load test*

i) Routine load test shall be conducted for test load of 1.50 times design load and measured settlement shall be as specified in IS:2911-Part-4. The procedure for testing shall be IS:2911-Part-4 or Bi–directional pile load test as given in **Appendix-7**. Routine load test should be done at locations of appropriate foundations of bridge to reconfirm or modify the allowable loads. Vertical and horizontal load tests should be properly designed to cover particular pile group. The lateral load test may be conducted on two adjacent piles. However, results of routine load tests shall not be used for upward revision of design capacity of piles. The minimum number of tests to be conducted is as given below for confirming pile capacity.

Total number of Piles for the Bridge	Minimum No. of Test Piles
Upto 50	2
Upto 150	3
Beyond 150	2 Percent of total piles (fractional number rounded to next higher integer number)

Note:

i) The number of tests may be judiciously increased depending upon the variability of foundations strata.

ii) Permissible Over Load

While conducting routine test on one of the pile belonging to a pile group, if the pile is found to be deficient (based on the settlement criteria at 1.5 times the design load) an overload upto 10percent of the reduced capacity may be allowed.

iii) For a quick assessment of pile capacity, high-strain dynamic tests may be conducted after establishing co-relations using the results of load tests. However, results of the strain dynamic tests shall not be used for upward revision of design capacity of pile. Detailed guidelines are at **Appendix-6 Part-1**. These methods can be followed.

iv) To have a fairly good idea about the quality of concrete and construction defects like voids, discontinuities, etc., pile integrity tests shall be conducted. Detailed guidelines and references are at **Appendix-6 Part 2**.

709.3 Geotechnical Capacity of Pile

709.3.1 *Resistance to vertical loads*

For calculating designed capacity of pile for load combination I of Clause 706.1.1, recommendation of **Appendix-5** should be followed. For calculating capacity of pile group, refer Clauses 709.3.3 and 709.3.4 and the allowable settlement criteria as per Clause 709.1.8. For purpose of these Clauses the following definitions will apply.

a) Cohesive soil (clay or plastic silt with Su ≤ 0.25 MPa);

b) Granular soil (sand, gravel or non-plastic silt with N (average within layer) 150 blows10.3 m (50 blows130 cms) ;

c) Intermediate Geometrical

Cohesive: e.g. clay shales or mudstones with 0.25 MPa (2.5 tsf) < Su< 2.5 MPa.

Cohesion less: e.g. granular tills, granular residual soils N>50 blows/0.3m (50 blows 130 cm);

d) Rock: Cohesive, Cemented Geomaterial with Su ≥ 2.5 MPa (25 tsf) or qu ≥ 5.0 MPa.

709.3.2 *Factor of safety*

The minimum factor of safety on ultimate axial capacity computed on the basis of static formula shall be 2.5 for load combination I of Clause 706.1.1 for piles in soil. For piles in rock, factor of safety shall be 3 on end bearing component and 6 on socket side resistance component, for load combination I of Clause 706.1.1 subject to further limits as stipulated in **Appendix-5**.

709.3.3 *Capacity of piles/group action*

The axial capacity of a group of piles should be determined by a factor to be applied to the capacity of individual piles multiplied by the number of piles of the group.

i) Factor may be taken as 1 in case of purely end bearing piles having minimum spacing of 2.0 times the diameter of pile and for frictional piles having spacing of minimum 3 times diameter of pile.

ii) For pile groups in clays, the group capacity shall be lesser of the following:

a) Sum of the capacities of the individual piles in the group.

b) The capacity of the group based on block failure concept, where the ultimate load carrying capacity of the block enclosing the piles in estimated.

709.3.4 *Settlement of pile group*

709.3.4.1 The capacity of a pile group is also governed by settlement criterion. Settlement of a pile group may be computed on the basis of following recommendations or by any other rational method.

709.3.4.2 Settlement of pile group in sands

The settlement of a pile group is affected by the shape and size of group, length, spacing and method of installation of piles. Methods given in IS:8009 (Part-II) of any other rational method may be used. The settlement of group of piles in sands can be calculated by assuming that the load carried by the pile group is transferred to the soil through an equivalent raft located at one third of the pile length upwards from the pile toe for friction piles. For end bearing piles the settlement can be calculated by assuming the raft placed at the toe of t'ıe pile group.

709.3.4.3 Settlement of pile group in clays

The settlement of pile group in homogeneous clays shall be evaluated using Terzaghi and Peck Approach which assumes that the load carried by the pile group is transferred to the soil through an equivalent footing located at one third of the pile length upwards from the pile toe. The load under the equivalent footing is assumed to spread into soil at a slope of 2 (vertical): I (horizontal).

The settlement for equivalent footing shall be evaluated in accordance with IS:8009 (Part-II).

709.3.4.4 Settlement of pile group in rock

Settlement of piles founded in rock may be computed as per IS:8009 (Part II) considering the value of in-situ modulus of rock mass.

709.3.5 *Resistance to lateral loads*

709.3.5.1 The ultimate lateral resistance of a group of vertical piles may be taken as the passive pressure acting on the enclosed area of the piles. Such passive pressure may be calculated over an equivalent wall of depth equal to 6D and width equal to L+ 2B.

Where,

D = Diameter of pile

L = Length between outer faces of pile group in plan perpendicular to direction of movement.

B = Width between outer faces of pile group in plan parallel to the direction of movement.

The minimum factor of safety on ultimate lateral resistance shall be 2.5.

709.3.5.2 The safe lateral resistance of pile-group must not exceed the sum of lateral resistance of individual piles. The safe lateral load resistance of individual pile depends on the modulus of horizontal sub-grade reaction of the foundation material as well as the structural rigidity of pile. Appropriate rational method of analysis using soil modulus as recommended by IS: 2911 may be used to calculate the same. For safe lateral load resistance in load combination I of Clause 706.1.1, the deflection at scour level shall not be greater than 1.0 percent of pile diameter. Checking of deflections in other load combinations is not required. For a group of vertical piles, confirmation of capacity of group by load testing is not required. For single pile the horizontal load test may be performed in accordance with IS: 2911. Testing shall be for free head condition for piles having free standing shaft above scour level upto the pile cap. For conduction test at scour level, it will be necessary to drive a larger diameter casing upto scour level so that the test pile above is free to deflect. The deflection at scour level may be measured directly, or may be calculated from deflection measured at higher (ground) level assuming that the pile acts as structural cantilever from the point of fixity. The point of fixity can be taken from the analysis performed for the design or calculated by simplified method given in IS: 2911.

709.3.6 *Uplift load carrying capacity*

709.3.6.1 Piles may be required to resist uplift forces of permanent or temporary nature when used in foundations subjected to large overturning moments or as anchorages in structures, like, underpasses subjected to hydrostatic uplift pressure.

709.3.6.2 The ultimate uplift capacity may be calculated with the expression of shaft resistance/ skin friction only, of the static formulae for compression loads and applying a reduction factor of 0.70 on the same. In case of rock, the socket length shall be measured from 0.3 m depth to actual depth of socket. The weight of the pile shall also be taken as acting against uplift. Pull out test shall be conducted for verification of uplift capacity. Factor of (2.5/0.7) = 3.5 on the ultimate strength shall be used.

709.3.6.3 The uplift capacity of pile group is lesser of the two following values:

- the sum of the uplift resistance of the individual piles in the group, and
- the sum of shear resistance mobilised on the surface perimeter of the group plus the effective weight of the soil and the piles enclosed in this surface perimeter.

709.3.6.4 Piles should be checked for structural adequacy against uplift forces together with other co-existent forces, if any.

709.3.6.5 The minimum factor of safety on ultimate uplift load calculated on the aforesaid basis shall be 2.5.

709.3.7 *Piles subjected to downward drag*

When a soil stratum through which pile shaft has penetrated into an underlying hard stratum compresses due to its own weight, or remoulding, or surface load etc., additional vertical load is generated along the pile shaft in such stratum. Such additional load coming on pile may be assessed on the following basis:

i) In the case of pile deriving its capacity mainly from friction, the value of downward drag force may be taken as 0.2 to 0.3 times undrained shear strength multiplied by the surface area of pile shaft embedded in compressible soil.

ii) In case of pile deriving its capacity mainly from end bearing, the value of downward drag force may be considered as 0.5 times undrained shear strength multiplied by the surface area of pile shaft embedded in Compressible soil.

iii) For a group of piles, the drag forces shall also be calculated considering the surface area of the block (i.e., perimeter of the group times depth) embedded in compressible soil. In the event of this value being higher than the number of piles in the group times the individual downward drag forces, the same shall be considered in the design.

iv) This reduction in capacity of pile is in the ultimate capacity.

709.4 Structural Design of Piles

Deleted. Refer IRC: 112

709.5 Design of Pile Cap

Deleted. Refer IRC:112

709.6 Important Consideration, Inspection/Precautions for Different Types of Piles

709.6.1 *Driven cast-in-situ piles*

709.6.1.1 Except otherwise stated in this code, guidance is to be obtained from IS:2911 (Part 1/ Section I).

709.6.1.2 The pile shoes which may be either of cast iron conical type or of mild steel flat type should have double reams for proper seating of the removable casing tube inside the space between the reams.

709.6.1.3 Before commencement of pouring of concrete, it should be ensured that there is no ingress of water in the casing tube from the bottom. Further adequate control during withdrawal of the casing tube is essential so as to maintain sufficient head of concrete inside the casing tube at all stages of withdrawal.

709.6.1.4 Concrete in piles shall be cast upto a minimum height of 600 mm above the designed top level of pile, which shall be stripped off to obtain sound concrete either before final set or after 3 days.

709.6.2 *Bored cast-in-situ piles*

709.6.2.1 The drilling mud, such as, bentonite suspension shall be maintained at a level

sufficiently above the surrounding ground water level to ensure the stability of the strata which is being penetrated throughout the boring process until the pile has been concreted.

709.6.2.2 The bores must be washed by fresh bentonite solution flushing to ensure clean bottom at two stages prior to concreting. Bores shall be cleaned by vaccum/pressure pumps so that all types of debris removed from bottom of pile to avoid soft toe condition. Bottom shall be examined by different shaft quantitative inspection devices commercially available. As a quality Control measure digital shaft area profile evaluator shall be used to calculate theoretical volume of bore and can be compared with actual concrete consumed.

709.6.2.3 Removal of concrete above cut-off level

It is desirable that the concrete above cut-off level is removed before the concrete is set. The concrete may be removed manually or by specially made bailer or other device. Such removal of concrete helps in preventing the damages of the good concrete below the cut-off level which results from chipping by percussion method.

The removal of concrete can be within ± 25 mm from the specified cut-off level preferably on the (-) side. On removal of the such concrete, the concrete should be compacted with rammer with spikes or it shall be vibrated.

In case the concrete is not removed before setting, a groove shall be made on outer perimeter by rotary equipment before chipping by percussion method.

709.6.3 *Driven precast concrete piles*

709.6.3.1 Except otherwise stated in this code, guidance is to be obtained from IS 2911 (Part I/ Section 3).

709.6.3.2 This type of piles for bridges may be adopted when length of pile as per design requirement is known with reasonable degree of accuracy. Extra length of pile may be cast to avoid lengthening of piles as far as possible. When unavoidable, the splicing for lengthening of steel may be used only after the method of splicing is tested and approved earlier.

The longitudinal reinforcement shall be joined by welding or by mechanical couplers. The concrete at top of original pile shall be cut down to sufficient length to avoid spelling by heat of welding. Location of mechanical couplers in neighboring reinforcement shall be such as to permit concreting between the bars.

709.6.3.3 During installation of piles, the final set or penetration of piles per blow of hammer should be checked taking an average of last 100 blows.

710 SUBSTRUCTURE

710.1 General

710.1.1 *Deleted. Refer IRC:112*

710.1.2 For the design of substructure below the level of the top of bed block, the live load impact shall be modified by the factors given below:

i) For calculating the pressure at the bottom surface of the pier/abutment cap 0.5

ii) For calculating pressure on the top 3 m of substructure below pier/abutment cap, decreasing uniformly from 0.5 to zero

iii) For calculating the pressure on the portion Zero of the substructure, at more than 3 m below the pier/abutment cap.

710.1.3 Structures designed to retain earthfill shall be proportioned to withstand pressure calculated in accordance with any rational theory. No structure shall, however, be designed to withstand a horizontal pressure less than that exerted by a fluid weighing 480 kg/m^3 , in addition to the live load surcharge, if any.

710.1.4 The backfill behind the wing wall, return walls and abutment shall conform to the specifications in **Appendix-8** with provision for proper drainage.

710.2 Piers

710.2.1 Piers in stream and channel should be located to meet navigational clearance requirements and give a minimum interference to flood flow. In general, piers should be placed parallel with the direction of stream current at flood stage. Piers in other locations, like, viaducts or land spans should be according to the requirement of the obstacles to cross over.

710.2.2 Where necessary, piers shall be provided at both ends with suitably shaped cut waters as given in IRC:6. However, cut and ease water where provided shall extend upto affluxed H.F.L. or higher, if necessary, from consideration of local conditions, like, waves, etc.

710.2.3 Pier may be in PSC, RCC, PCC or masonry. Only solid section should be adopted for masonry piers. The design of masonry piers should be based on permissible stresses as provided in IRC: 40.

710.2.4 The thickness of the walls of hollow concrete piers should not be less than 300 mm.

710.2.5 The multi-column piers of bridges across rivers carrying floating debris, trees or timber should be braced throughout the height of the piers by diaphragm wall of minimum 200 mm thickness. Unbraced multiple column piers may be allowed depending upon the performance of similar structures in similar conditions of river. However, type and spacing of such bracing, when adopted, shall be predetermined.

710.2.6 Piers shall be designed to withstand the load and forces transferred from the superstructure and the load and forces on the pier itself, apart from the effect of its self-weight. In general, pier may be solid, hollow or framed structures.

710.2.7 In case of pier consisting of two or more columns, the horizontal forces at the bearing be distributed on columns as required by appropriate analysis.

710.2.8 If the piers consist of either multiple piles or trestle columns spaced closer than three times the width of piles/columns across the direction of flow, the group shall be treated as a solid pier of the same overall width and the value of K taken as 1.25 for working out pressure due to water current according to relevant Clause 213.7 of IRC: 6. If such piles/columns are braced then the group should be considered a solid pier irrespective of the spacing of the columns.

710.2.9 Hollow piers shall be provided with suitably located weep-holes of 100 mm diameter for enabling free flow of water to equalise the water levels on inside and outside; considering rate

of rise/fall of flood/tide water. The pier walls should be checked for expected differential water-head/wave pressure and silt pressure. In absence of detailed calculations, a minimum difference of 1.5 m in water levels on two sides shall be assumed.

710.2.10 The lateral reinforcement of the walls of hollow circular RCC pier shall not be less than 0.3 percent of the sectional area of the walls of the pier. This lateral reinforcement shall be distributed 60 percent on outer face and 40 percent on inner face.

710.3 Wall Piers

Deleted. Refer IRC: 112

710.4 Abutments

710.4.1 The abutments will carry superstructure from one side. It should be designed/ dimensioned to retain earth from the approach embankment.

710.4.2 The abutments should be designed to withstand earth pressure in normal condition in addition to load and forces transferred from superstructure. In addition, any load acting on the abutment itself, including self-weight, is to be considered.

710.4.3 In case of spill through type abutment, the active pressure calculated on the width of the column shall be increased by 50 percent where two columns have been provided and by 100 percent where more than two columns have been provided.

710.4.4 All abutments and abutment columns shall be designed for a live load surcharge equivalent to 1.2 m height of earth fill. The effective width of the columns need not be increased as in Clause 710.4.3 for surcharge effect when spill through abutment is adopted.

710.4.5 Abutment should also be designed for water current forces during 'scour all round' condition.

710.4.6 The abutment may be of plain or reinforced concrete or of masonry. The abutment may be either solid type, buttressed type, counterfort type, box type or spill through type. For spill through abutment, column type or wall type, analysis may be carried out as for piers. Counterfort type abutment may be treated as T or L type as the case may be and the slab may be designed as continuous over counterforts.

710.4.7 *Deleted. Refer IRC:6*

710.4.8 The weight of earth filling material on heel may be considered. In case of toe, the weight may be considered if the bed is protected.

710.4.9 In case of spill through type abutment, it should be ensured that the slope in front of the abutment is well protected by means of suitably designed stone pitching and launching aprons.

710.4.10 In case of abutments having counterfort, the minimum thickness of the front wall should not be less than 200 mm and the thickness of the counterfort should not be less than 250 mm.

710.4.11 In case of box type abutments, weep holes shall be provided similar to hollow piers as per Clause 710.2.9.

710.5 Intermediate Heavy Pier

710.5.1 Intermediate Heavy Pier may have to be provided at locations where there may be a need of increasing waterway subsequently. The design of such Intermediate Heavy Pier shall be such that it should be possible to convert them to the similar shape as piers in the active channel.

710.5.2 For multiple span arch bridges, Intermediate heavy piers shall be provided after every fifth span or closer. It is designed for condition that even if arch on one side of it collapses, the pier and arches on other side will remain safe.

710.6 Dirt Walls, Wing Walls and Return Walls

710.6.1 Wing walls shall be of sufficient length to retain the roadway to the required extent and to provide protection against erosion.

710.6.2 A dirt wall shall be provided to prevent the earth from approaches spilling on the bearings. A screen wail of sufficient depth (extended for at least 500 mm depth into the fill) to prevent slipping of the backfill in case the abutment is of the spill through type, shall be provided.

710.6.3 The wing walls may be of solid type. The return walls may be of solid or counterfort type. The material used may be plain or reinforced concrete or masonry.

710.6.4 Dirt wall/ballast wall and screen wall shall be provided with minimum thickness of 200 mm.

710.6.5 The wing walls should be designed primarily to withstand the earth pressure in addition to self-weight.

710.6.6 The top of the wing/return walls shall be carried above the top of embankment by at least 100 mm to prevent any soil from being blown or washed away by rain over its top. A drainage arrangement for return wall/wing wall may be provided similar to that for the abutment specified in **Appendix-8**.

710.6.7 The cantilever returns where adopted should not be more than 4 metres long.

710.6.8 In case of open foundations, wing and return walls should be provided with separate foundations with a joint at their junction with the abutment.

710.6.9 Wing walls may be laid at any suitable angle to the abutment. In case of river bridges, these are normally splayed in plan at 45°. The return walls may be provided at right angles to the abutment. Return walls shall be designed to withstand a live-load surcharge equivalent to 1 .2 m height of earthfill.

710.6.10 The box type return wall at right angles at both ends of the abutments connected by wall type diaphragm may be adopted where found suitable. However, in such cases, no reduction in the earth pressure for the design of the abutment should be considered. The top of diaphragm should slope inwards to the centre of carriageway for facilitating proper rolling of the embankment behind the abutment.

710.6.11 Solid type of wing/return walls on independent foundations can be suitably stepped up towards the approaches depending upon the pattern of scour, local ground conditions and its profile, safe bearing capacity, etc.

710.6.12 In case of wing walls or return walls, the foundation shall be taken adequately into the firm soil.

710.7 Retaining Walls

710.7.1 The minimum thickness of reinforced concrete retaining wall shall be 200 mm.

***710.7.2* to 7.10.7.4 Deleted Refer IRC: 112**

710.8 Pier and Abutment Caps

710.8.1 The width of the abutment and pier caps shall be sufficient to accommodate:

i) The bearings leaving an offset of 150 mm beyond them,

ii) The ballast wall.

iii) The space for jacks to lift the superstructure for repair/replacement of bearings, etc.

iv) The equipment for prestressing operations where necessary, over and above space for end block in cast in-situ cases.

v) The drainage arrangement for the water on the cap.

vi) Seismic arrestor, if provided

vii) To accommodate inspection ladders

710.8.2 The thickness of cap over the hollow pier or column type of abutment should not be less than 250 mm but in case of solid plain or reinforced concrete pier and abutment, the thickness can be reduced to 200 mm.

710.8.3 Pier/Abutment caps should be suitably designed and reinforced to take care of concentrated point loads dispersing in pier/abutment. Caps cantilevering out from the supports or resting on two or more columns shall be designed to cater for the lifting of superstructure on jacks for repair/replacement of bearings. The locations of jacks shall be predetermined and permanently marked on the caps.

710.8.4 In case bearings are placed centrally over the columns and the width of bearings/ pedestals is located within half the depth of cap from any external face of the columns, the load from bearings will be considered to have been directly transferred to columns and the cap beam need not be designed for flexure.

710.8.5 The thickness of the cap over masonry piers or abutment shall not be less than 500 mm. The minimum width at the top of such piers and abutments of slab and girder bridges just below the caps shall be as given below:

Span in meters	3m	6m	12m	24 m
Top width of pier carrying simply supported spans in m	0.50	1.0	1.2	1.6
Top width of abutment and of piers carrying continuous spans in m	0.40	0.75	1.0	1.3

710.8.6 Except the portion under bearings, the top surface of caps should have suitable slope in order to allow drainage of water.

710.8.7 *Deleted. Refer IRC: 112*

710.9 Cantilever Cap of Abutment and Pier

Deleted. Refer IRC: 112

710.10 Pedestals below Bearing

710.10.1 The pedestals should be so proportioned that a clear offset of 150 mm beyond the edges of bearings is available.

710.10.2 The height of the pedestal should be between 150 mm and 500 mm. Where the depths of superstructures from two adjacent spans on a common pier differ and require use of pedestals of more height below one of the spans, the shape of pier cap or the diaphragm of superstructure shall be modified to restrict the height of pedestals to 500 mm. For pedestals whose height is less than its width, the requirement of the longitudinal reinforcement as specified for short column need not be insisted upon.

710.10.3 *Deleted. Refer IRC: 112*

710.10.4 *Deleted. Refer IRC: 112*

Appendix-1
(Clause 704.3)

GUIDELINES FOR SUB-SURFACE EXPLORATION

The objective of sub-surface exploration is to determine the suitability of the soil and/or rock for the foundation of bridges and approaches. The sub-surface exploration for bridges is carried out in two stages namely preliminary and detailed. It may require additional/confirmatory exploration during construction stage.

For geophysical investigation, a reference may be made to IRC: 123.

For preliminary and detailed sub-surface investigation only rotary drills shall be used. The casing shall also be, invariably provided with diameters not less than 150 mm upto the level of rock if any. However, use of percussion or wash boring equipment shall be permitted only to penetrate through boulder or gravelly strata for progressing the boring but not for collection of samples. While conducting detailed borings, the resistance to the speed of drilling, i.e., rate of penetration, core loss, etc. shall be carefully recorded and presented in "Borelog chart and data sheet" to evaluate the different types of strata and distinguish specially sand from sandstone, clay from shale, etc.

For preliminary and detailed sub-surface investigation, only double tube diamond drilling method shall be used. In soft and weak rocks such as tuffs, soft shales, etc. where rock core recovery is less than 20%, consideration may be given to use of triple tube diamond drilling on major projects under the instruction of the Engineer-in-charge.

2. PRELIMINARY INVESTIGATION

2.1 Preliminary investigation shall include the study of existing geological information, previous site reports, geological maps, etc. and surface geological examination. These will help to narrow down the number of sites under consideration, minimum depth of investigation and also to locate the most desirable location for detailed sub-surface investigation.

3 DETAILED INVESTIGATION

3.1 Based on data obtained after preliminary investigations, the bridge site, the type of structure with span arrangement and the location and type of foundations, the programme of detailed investigations, etc. shall be tentatively decided. Thereafter the scope of detailed investigation including the extent of exploration, number of bore holes, type of tests, number of tests, etc. shall be decided in close liaison with the design engineer and the exploration team, so that adequate data considered necessary for detailed design and execution are obtained.

3.2 The exploration shall cover the entire length of the bridge and also at either end a distance of zone of influence, i.e. about twice the depth below bed of the last main foundation to assess the effect of the approach embankment on the end foundations. Generally, the sub-surface investigations in erodible soil should be extended to minimum 15 m below the anticipated founding level. In case of well foundation, sub-surface investigations should extend to at least of 2 times the proposed diameter of well or 15 m below the anticipated founding level whichever is higher. In case of pile foundation in hard rock, sub-surface investigations should extend to minimum depth of 4 times the diameter of pile below the pile tip. The sub-surface investigations in

non-erodible strata shall conform with Table 2 of Section 8.6. However, where such investigations end in any unsuitable or questionable foundation material, the exploration shall be extended to a sufficient depth into firm and stable soils or to rock.

3.2.1 Minimum Number of Bore Holes Minimum number of bore holes shall be as below:

3.2.1.1 Bridges

i) For small bridges length 6 m to 30m

a) One at each abutment, and

b) One at any pier location

ii) For bridges length more than 30 m

a) One at each foundation location

3.2.1.2 Culverts

Open trench 3 m deep from lowest bed level

3.2.1.3 Box type structures

Open trench 3 m deep from lowest bed level at every 10 m interval

3.2.1.4 Retaining Wall

One bore at every 50 m length

3.2.1.5 Embankment more than 6 m high

One bore hole at every 100 m interval

3.2.2 *Additional drill holes:* Where the data made available by detailed exploration indicate appreciable variation specially in case of foundations resting on rock, it will be necessary to resort to additional drill holes to establish a complete profile of the underlying strata. Location and depth of additional drill holes will have to be decided depending upon the extent of variation in local geology and in consultation with design engineer.

3.3 The scope of the detailed sub-surface exploration shall be fixed as mentioned in para 3.1 and 3.2. However, as a general guide it shall be comprehensive enough to enable the designer to estimate or determine the following:

i) engineering properties of the soil/rock;

ii) location and extent of weak layers and cavities, if any, below hard founding strata;

iii) the sub-surface geological condition, such as, type of rock, structure of rock, i.e. folds, faults, fissures, shears, fractures, joints, dykes and subsidence due to mining or presence of cavities;

iv) ground water level;

v) artesian, conditions, if any,

vi) quality of water in contact with the foundation;

vii) depth and extent of scour;

viii) suitable foundation level;

ix) safe bearing capacity of foundation stratum;

x) probable settlement and probable differential settlement of the foundations;

xi) likely sinking or driving effort;

xii) likely construction difficulties; and

xiii) Possibility of liquefaction of any sub-surface strata.

4 CONSTRUCTION STAGE EXPLORATION

Such exploration may become necessary to verify the actually met strata vis-à-vis detailed investigation stage or when a change in the sub-soil strata/rock profile is encountered during construction. In such situations, it may be essential to resort to further explorations to establish the correct data, for further decisions.

5 METHOD OF TAKING SOIL SAMPLES

The size of the bores shall be predetermined so that undisturbed samples as required for the various types of tests are obtained. The method of taking samples shall be as given in IS:1892 and IS:2132. The tests on soil samples shall be conducted as per relevant part of IS:2720.

6 DETAILS OF EXPLORATIION FOR FOUNDATIONS RESTING ON SOIL (ERODIBLE STRATA)

6.1 The type and extent of exploration shall be divided into the following groups keeping in view the different requirements of foundation design and the likely method of data collection:

i) Foundations requiring shallow depth of exploration;

ii) Foundations requiring large depth of exploration; and

iii) Fills behind abutments and protective works.

6.2 Foundations Requiring Shallow Depth of Exploration (Open Foundation)

These shall cover cases where the depth of exploration is not large and it is possible to take samples from shallow pits or conduct direct tests, like plate load tests, etc. This will also cover generally the foundation soil for approach embankments, protective works, etc.

6.2.1 The primary requirements are stability and settlement, for which shearing strength characteristics, load settlement characteristics, etc. need determination

6.2.2 Tests shall be conducted on undisturbed representative samples, which may be obtained from open pits. The use of plate load test (IS 1888-Method of Load Test on Soils) is considered desirable for ascertaining the safe bearing pressure and settlement characteristics.

A few exploratory bore holes or soundings shall be made to safeguard against presence of weak strata underlying the foundation. This shall extend to 15m below proposed/anticipated founding level.

Note: For better interpretation, it will be desirable to correlate the laboratory results with the in-situ tests, like plate load tests, penetration test results.

6.2.3 The tests to be conducted for properties of soil are different for cohesive and cohesion less soils. These are indicated below. While selecting the tests and interpreting the results, limitations of applicability of chosen tests shall be taken into account. A most suitable and appropriate combination of these shall be chosen, depending on the properties needed for design and constructional aspects.

I) Cohesionless Soil

a) Laboratory Tests

i) Classification tests, index tests, density determination etc.

ii) Direct shear strength of cohesionless soil sample compacted to required densities.

b) Field Tests

i) Plate Load Test, (as per IS:1888)

ii) Standard Penetration Tests (as per IS:2131)

iii) Dynamic Cone Penetration Test, (as per IS:4968 Part I or Part II)

iv) Static Cone Penetration Test (as per IS: 4968 Part III)

v) Pressuremeter/Dilatometer test (as per international codes)

Note: Where dewatering is expected, permeability tests may be conducted (as per IS:2720 Part XVII).

II) Cohesive Soil

a) Laboratory Tests

i) Classification Tests, Index Tests, Density Determination etc.

ii) Shear strengths by triaxial/direct shear, etc.

iii) Unconfined Compression Test (IS:2720 Part X)

iv) Consolidation Test (IS:2720 Part V)

b) Field Tests

i) Plate Load Test, (as per IS:1888)

ii) Vane Shear Test (as per IS:4434)

iii) Static Cone Penetration Test, (as per IS:4968 Part !II),

iv) Standard Penetration Test (as per IS:2131)

v) Dynamic Cone Penetration Test (as per IS:4968

Notes: 1. Where dewatering is expected, permeability tests may be conducted (as per IS: 2720 Part XVII).

2. Where liquefaction during earthquakes is critical, energy transferred through the SPT hammer shall be measured so as to correct the SPT N-value to standardized 60% energy transfer.

6.3 Foundations Requiring Large Depth of Exploration

6.3.1. In this group are covered cases of deep wells, pile foundations etc. where the use of boring equipment, special techniques of sampling, in-situ testing, e.c. become essential. In addition to the problems of soil and foundation interaction an important consideration can be the soil data required from construction considerations. Often in the case of cohesionless soils, undisturbed samples cannot be taken and recourse has to be made to in-situ field tests.

Boring and sampling tends to cause remoulding of sensitive clays. Also for fissured or layered clays, the sample may not truly represent the in-situ properties, due to disturbance and stress changes caused by boring and sampling activity. In such cases, in-situ tests shall be performed.

6.3.2 The sub-surface exploration can be divided into three zones:

i) Between bed level and upto anticipated maximum scour depth (below H.F.L.)

ii) From the maximum scour depth to the foundation level; and

iii) 15 m depth below proposed founding level.

6.3.3 Sampling and testing (in-situ and laboratory) requirement will vary in each case and hence are required to be assessed and decided from case-to-case. The sub-soil water shall be tested for chemical properties to evaluate the hazard of deterioration to foundations. Where dewatering is expected to be required, permeability characteristics should be determined.

6.3.4 For the different zones categorized in Para **6.3.2**, the data required, method of sampling, testing, etc. are given in **Table 1.** Samples of soils in all cases shall be collected at every 1 to 1½ metre or at change of strata.

Table 1 Sub-Soil Data Required for Deep Foundations

Zone	Data/Characteristics Required	Sampling and Testing
Bed level to anticipated maximum scour depth/depth to which liquefaction may occur	i) Soil Classification ii) Particles size distribution iii) Permeability, where dewatering is expected	**Sampling –** Disturbed samples may be collected **Laboratory Tests** Classification Tests, including particle size distribution **In-situ Tests:** Permeability tests
i) Maximum anticipated scour level to the foundation level	i) Soil Classification ii) Particles size distribution iii) Moisture content, density, void ratio, iv) Shear strength v) Compressibility vi) Permeability where dewatering is expected vii) Chemical analysis of soil and ground water (for aggressive elements)	**Sampling for Laboratory Tests:** Undisturbed samples shall be collected for these tests. As an exception, for i) and ii) disturbed samples may be permitted. **Laboratory Tests:** a) Classification Tests including particle size distribution, b) Moisture content, density , void ratio; c) Shear strength – Triaxial tests to be done on undisturbed samples – Unconfined compression tests to be done on undisturbed and/ or remoulded samples d) Consolidation tests. **In-situ Tests for Cohesionless soils -.** a) Dynamic Cone Penetration Test b) Standard Penetration Test c) Down hole/Cross hole seismic survey d) Permeability tests **In-situ Tests for Cohesive Soils -** a) Dynamic Cone Penetration Test b) Static Cone Penetration Test – cone and skin resistance c) Field Vane Shear Test d) Permeability tests e) Down hole/Cross hole seismic surveys
Foundation level to minimum 15m below it	i) Soil Classification ii) Shear Strength iii) Compressibility	**Same as above**

Notes:

i) Laboratory tests to be conducted according to the relevant parts of IS:2720

ii) Use of sophisticated equipment like pressure meter may be made, if suitable co-relations for interpretation of data collected are available.

iii) Down hole/Cross hole seismic surveys shall be as per ASTM D 4428/D 4428 M

iv) Seismic Methods and/or Electrical Resistivity Method can be used for soil/rock profiling. Down hole/Cross hole seismic surveys could be used for establishing elastic moduli and rock profiling a greater depths.

Down hole/Cross hole seismic surveys are useful for long bridges (i.e. of the order of 1 km and above) and for reducing the number of boxes taken in the portions that are permanently under water. For these applications, geotechnical profiles obtained by seismic methods shall be calibrated/confirmed with actual profiles taken by bores at intermediate locations.

6.4 Fill Material

Representative disturbed samples shall be collected from the borrow pit areas. Laboratory tests shall be conducted for determining the following:

i) Classification and particle size

ii) Moisture content

iii) Density vs. moisture content relationship

iv) Shearing strength

v) Permeability

Note: The shearing strength shall be obtained for the density corresponding to the proposed ensity for the fill.

7. EXPLORATIION FOR FOUNDATIONS IN LIQUEFIABLE SOILS

In addition to the requirements in Para 6 above, geotechnical investigations in liquefiable soils should preferably include the following:

i) Measurement of energy transferred to SPT sampler: This can help assess liquefaction better.

ii) Static Cone penetration tests with or without pore water pressure measurement.

iii) Shear wave velocity determination by geophysical tests such as cross-hole tests, SASW /MASW tests (See IRC 123).

8. DETAILS OF EXPLORATION FOR FOUNDATION RESTING ON ROCK

8.1 Basic Information required from Explorations

i) Geological system;

ii) Depth of rock and its variation over the site;

iii) Whether isolated boulder or massive rock formation;

iv) Extent and character of weathered zone;

v) The structure of rock including bedding planes, faults, etc.;

vi) Properties of rock material, like, strength, geological formation, etc.;

vii) Quality and quantity of returning drill water; and

viii) Erodibility of rock to the extent possible, where relevant.

ix) Rock Quality Designation

8.2 Exploration Program

If preliminary investigations have revealed presence of rock within levels where the foundation is to rest, it is essential to take up detailed investigation to collect necessary information mentioned in the preceding para.

8.2.1 The extent of exploration shall be adequate enough to give a complete picture of the rock profile, both in depth and across the channel width, for assessing the constructional difficulties in reaching the foundation levels. Keeping this in view, it shall be possible to decide the type of foundations, the construction method to be adopted for a particular bridge, the extent of even seating and embedment into rock of the foundations. It is desirable to take at least one drill hole per pier and abutment and one on each side beyond abutments.

8.2.2 The depth of boring in rock depends primarily on local geology, erodibility of the rock, the extent of structural loads to be transferred to foundation, etc. Normally, it shall pass through the upper weathered or otherwise weak zone, well into the sound rock. The minimum depth of drilling shall be referred to **Table 2** of Section 8.6.

8.2.3 Drilling through rock shall normally be done using a hydraulic rotary core drilling rig. Use of calyx drilling shall not be permitted except under unusual conditions such as sites with difficult access or other difficult logistics issues with the permission of the Engineer-in-charge.

8.3 Detailed Investigations for Rock at Surface or at Shallow Depths

In case of rock at shallow depths which can be conveniently reached, test pits or trenches are the most dependable and valuable methods, since they permit a direct examination of the surface, the weathered zone and presence of any discontinuities. For guidance, IS:4453-Code of Practice for exploration by pits, trenches, drafts and shafts may be referred to. In case of structurally disturbed rocks, in-situ tests may be made in accordance with IS:7292-Code of Practice for in-situ determination of rock properties by flat jack, IS:7317-Code of Practice for Uni-axial Jacking Test for Deformation Modulus and IS:7746-Code of Practice for in-situ Shear Test on Rock.

8.4 Detailed Investigation for Rock at Large Depths

8.4.1 Details of rock depth and characteristics along various lines shall be determined by appropriate geophysical investigations. This covers cases where recourse is to be made to sounding, boring and drilling. An adequate investigation program has to be planned to cover the whole area for general characteristics and in particular the foundation location, for obtaining definite information regarding rock depth and its variation over the foundation area. The detailed program of exploration will depend on the type and depth of overburden, the size and importance of the structure, etc. To decide this, geophysical methods adopted at the preliminary investigation stage may be helpful.

8.4.2 The investigation of the overburden soil layers shall be done as per details given for the foundations resting in soil. However, in case of foundations resting on rock, tests on overburden shall be carried out only when necessary, e.g., foundation level lower than scour levels.

8.4.3 The core shall be stored properly in accordance with IS: 4078-Code of Practice for Indexing and Storage of Drill Cores. Wherever triple tube core barrel in used, inner most transparent plastic tube shall be stored directly in core box with sample.

8.4.4 The rock cores obtained shall be subjected to tests to get necessary data for design as follows:

i) Visual identification for
   a) Texture,
   b) Structure
   c) Composition
   d) Colour
   e) Grain size
   f) Petrography

ii) Laboratory tests may be done for
   a) Specific gravity
   b) Porosity
   c) Water absorption
   d) Compressive strength

Note: Generally, shear strength tests will suffice for design purposes. Other tests may need to be done in special case. The shear strength tests can be done as unconfined compression, triaxial compression or direct shear test.

8.4.5 Use of in-situ tests for measuring strength and deformation characteristics may be made. Use of bore hole photography will be desirable to evaluate the presence of faults, fissures or cavities, etc. Particularly in weak and/or highly weathered rock, where triple tube core barrel is used for drilling, in-situ tests such as Standard Penetration Test and/or pressure meter test shall be conducted at every 1.5 m interval in the influence zone of footing or pile.

These in-situ tests are also useful in zones where water loss is recorded during drilling. However, selections of these tests have to be done judiciously and planned along with drilling operation.

8.5 Special Cases

8.5.1 Investigation for conglomerate

A drill hole shall be made same as for rock. The samples collected shall be subjected to suitable tests depending upon the material. Special care shall be taken to ascertain the credibility of the matrix.

8.5.2 Investigation for laterites

The investigation shall generally be similar to that required for cohesive soils. In case of hard laterite, recourse may have to be made to core drilling as for soft rocks.

8.5.3 Investigation of Soft Rock with poor Core Recovery

If drilling using double tube core barrel yields low core recovery (say <20%), use of triple tube core barrel may be considered on major projects if the Engineer-in-charge is of the opinion that core recovery and RQD values have a significant impact on the design SBC.

8.5.4 Investigation for boulder-gravel strata

Geotechnical investigations in boulder deposits pose a veritable challenge. Suggested methods include:

- Borehole by a percussion method
- Borehole by hydraulic rotary drilling
- Large diameter holes (400-500 mm diameter) using piling rigs
- DTH odex drilling combined with percussion / rotary drilling

The boreholes should be supplemented with geophysical tests and/or dynamic cone penetration test using a 65 mm diameter cone, Boulder-boulder or concrete-boulder horizontal shear tests and footing load tests

8.6 Termination Criteria for Boreholes drilled through Rock

For the purpose of field records and deciding the termination of borehole in rock, the criteria for field classification and respective minimum termination depth are indicated in Table 2.

Table 2: Criterion for field classification and corresponding termination depth in rock

Rock Type	Field Classification	Extrapolated N Value	Break ability	Penetration	Scratch	Termination Criteria
Extremely Strong Very Strong Strong	Hard rock	600 and above	Difficult to break against solid object with hammer	-	Cannot be scratched with knife	3 m
			Broken against solid object with hammer	-	Cannot be scratched with knife	
Moderately Strong Moderately Weak	Moderate rock	200-600	Broken in hand by hitting with hammer	-	Scratched with knife	5 m
Weak Very Weak	Weak rock	Less than 200	Broken by tapping on sample with hammer	-	Scratched with thumbnail	10 m

The depths given in Table 2 are recommended. Deeper boreholes may be required in some cases, particularly where piles are socketed into rock or well foundations are installed into the rock, or where rock is scourable. In such cases, the Engineer-in-charge may decide the depth of investigation into rock depending upon the project / structure requirements and other considerations.

9. CLASSIFICATION AND CHARACTERISTICS OF ROCKS

9.1 Identification and classification of rock types for engineering purposes may, in general be limited to broad, basic physical condition in accordance with accepted practice. Strength

of parent rock alone is of limited value because overall characteristics depend considerably on character, spacing and distributions of discontinuities of the rock mass, such as the joints, bedding, faults and weathered seams.

9.2 Classification of Rocks

Rocks may be classified based on their physical condition and Unconfined Compressive Strength as per **Table-3**.

Table-3 Classification of Rocks

Rock Type	Description	Unconfined Compressive Strength (UCS) in MPa
Extremely Strong	Cannot be scratched with knife or sharp pick. Breaking of specimen could be done by sledge hammer only.	>200
Very Strong	Cannot be scratched with knife or sharp pick. Breaking of specimens requires several hard blows of geologists' pick	100 to 200
Strong	Can be scratched with knife or pick with difficulty. Hard blow of hammer required to detach hand specimen	50 to 100
Moderately Strong	Can be scratched with knife or pick. 6 mm deep gouges or grooves can be made by hand blow of geologists' pick. Hand specimen can be detached by moderate blow	12.5 to 50
Moderately Weak	Can be grooved or gouged 1.5 mm deep by firm pressure on knife or pick point. Can be broken into pieces or chips of about 2.5 mm maximum size by hard blows of the points of geologists' pick.	5 to 12.5
Weak	Can be grooved or gouged easily with knife or pick point. Can be break down in chips to pieces several cm's in size by moderate blows of pick point. Small thin pieces can be broken by finger pressure.	1.25 to 5
Very Weak	Can be carved with knife. Can be broken easily with point of pick. Pieces 25 mm or more in thickness can be broken by finger pressure. Can be scratched easily by finger nail.	< 1.25

Note:

1) The Unconfined Compressive Strength values are as IS:9413 (Clause 44.2.6)

2) **Table 2** should not be used to infer the Unconfined Compressive Strength of rock. Actual laboratory test value of rock core should be used.

10. Presentation of Data

The presentation of data collected shall be done as illustrated in **Sheet No. 1** and **2**

R.L. OF GROUND BORE 75.00 (REF. LEVEL) — FIELD TEST RESULTS — DATE COMPLETED

(NOTE:- SEPARATE SHEET TO BE GIVEN FOR EACH BORE HOLE/PIT)

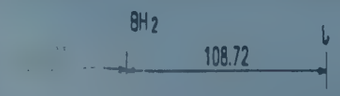
BORE HOLE No. 2

ELEVATION IN METRES	DEPTH IN METRES BELOW REFERENCE	DATE OF SAMPLING	NATURE OF SAMPLING	DEPTH OF SAMPLE BELOW REFERENCE LEVEL	SAMPLE REF. NO.	LEVEL OF WATER TABLE/ L.W.L	LEVEL AT WHICH ARTESIAN CONDITION EXPERIENCED IF ANY	ARTESIAN PRESSURE	S.P.T BLOWS PER 30cm: DEPTH IN METRES	S.P.T BLOWS PER 30cm: VALUE	STATIC PENETRATION RESISTANCE kg/cm²: POINT RESISTANCE	STATIC PENETRATION RESISTANCE kg/cm²: SKIN RESISTANCE	DIA OF CASING USED (mm)	TIME TAKEN FOR BORING FOR EACH 30cm IN MINUTES	PER CENT CORE RECOVERY	DESCRIPTION OF CORE ROCK TYPE COLOUR GRAIN SIZE TERTURE MINERAL COMPOSITION DEGREE OF WEATHERING	STRUCTURE DETAIL OF JOINTS BEDDING FAULTS FOLIATION VIEW SEEN CRUSHED ZONE	DETAILS OF WATER COLOUR ETC.	SIZE OF CORE PIECES	SIZE OF HOLE	WATER LOSS	IN SITU ROCK TESTS	SYMBOLIC REPRESENTATION	VISUAL DESCRIPTION OF SOIL	TYPE OF TEST CONDUCTED IN THE LABORATORY	% GRAVEL > 72mm	% SAND 2.00–0.06mm	% SILT 0.06 – 002mm	% CLAY < 0.002mm	DRY DENSITY GMS/cm3	SPECIFIC GRAVITY	VOID RATIO	NATURAL MOISTURE	UNCONFINED COMPRESSIVE STRENGTH kg/cm²: UNDISTURBED	UNCONFINED COMPRESSIVE STRENGTH kg/cm²: REMOULDED	SENSITIVITY	SHEARING STRENGTH CHARACTERISTIC: COHESION 'C' IN KG/cm²	SHEARING STRENGTH CHARACTERISTIC: ANGLE OF SHEARING RESISTANCE	COMPRESSION INDEX CC	PERCENTAGE SOLUBLE SALTS: CARBONATES AS No_2 Co_3	PERCENTAGE SOLUBLE SALTS: CHLORIDES AS No_2 Cl	PERCENTAGE SOLUBLE SALTS: SULPHATES AS No_2 So_4	ORGANIC MATTER %	ROCK CRUSHING STRENGTH IN KG/cm²	REMARKS
75	0					▼ RL 77.0																																							
		7-1-88	D	3	S-1/2		NIL		3.0	7			150													POORLY GRADED SAND		2.3	15.8	10.8	1.1	1.69			17.6												
70	5																																													
		8-1-88	UD	9	S-2/2		NIL		10.1	10			150													CLAY OF MEDIUM PLASTICITY WITH SOME KANKAR		3.7	3.6	38.9	43.6	1.69			17.7												STEPS TO BE TAKEN TO RECONCILE THE FIELD AND LABORATORY TEST DATA, WHEREVER POSSIBLE
65	10																																												
		11-1-88	UD	15	S-3/2		NIL		13.0	12			150																																	
		12-1-88	UD	19.5	S-4/2		NIL		17.6	17			100													CLAY OF HIGH PLASTICITY WITH SOME KANKAR		2.3	5.7	19.7	52.3	1.66			20.0	0.9	0.6	1.5								
55	20	12-1-88	UD	21.0	S-5/2		NIL						100																																	
50	25								24.2	23																																				
		15-1-88	UD	27.0	S-6/2		NIL						100													CLAY OF MEDIUM PLASTICITY WITH SOME KANKAR	UC	5.5	11.1	34.2	49.2	1.96	2.67	0.71	30.69	1.6	0.5	2.2	0.59	θ	NIL	NIL	0.85	40.02		
45	30	18-1-88	UD	30.1	S-7/2		NIL		31.0	24			100																																	
PROPOSED FOUNDATION LEVEL 42.70																																														
40	35	21-1-88	D	38.1	S-8/2		NIL		33.1	32			100													CLAY OF HIGH PLASTICITY WITH SOME KANKAR		9.1	7.3	34.9	48.7	1.70	2.64	0.55	22.89	2.85	1.9	1.4								
35	40								41.2	33			100																																	
30	45	24-1-88	UD	44.9	S-9/2		NIL						100																																	
		24-1-88	D	46.2	S-10/2		NIL						75	30	0	LIME STONE HEAVILY SHEARED WEATHERING			25mm							POORLY GRADED SAND		13.9	0.5	5.6	1.66	2.64	0.39	14.85						NIL	2.9	NIL			
25	50	25-1-88	UD	50.5	S-11/2		NIL						75	55	35	LIME STONE MODERATE WEATHERING SHEARED	VERY CLOSE JOINTS SMALL CAVITIES		125mm																													
																										LIME STONE																		69.0		
20	55	27-1-88	UD	56.0	S-12/2		NIL						75	42	47	SAND STONE GRAINED POORLY CEMENTED SHEARED			157mm																													
15	60	28-1-88	UD	60.5	S-13/2								75	61	100	SAND STONE WITH STREAKS OF SILT STONE DENSE WELL CEMENTED			212mm							SAND STONE																		57.0		

NOTES:-

1. Classification of soil shall be as per IS:1498
2. Abbreviation used
   D = Disturbed sample
   UD = Undisturbed sample
   S-1/2 Sample No.1 at Bore Hole No.2
3. Type of Machine used in Boring
4. Type of Core Barrel Used ----------
5. Size of Samples ---------
6. Position of Ground water table shown thus.
7. Shearing Strength Characteristics based on Triaxial/Box Shear Test on Undisturbed/Disturbed Sample At ---------- gm/cm³ Density Trioxial Test Quick Shear/Cosolidated Quick/ Slow.

POSITION OF BORE HOLE WITH REFERENCE TO REFERENCE POINT

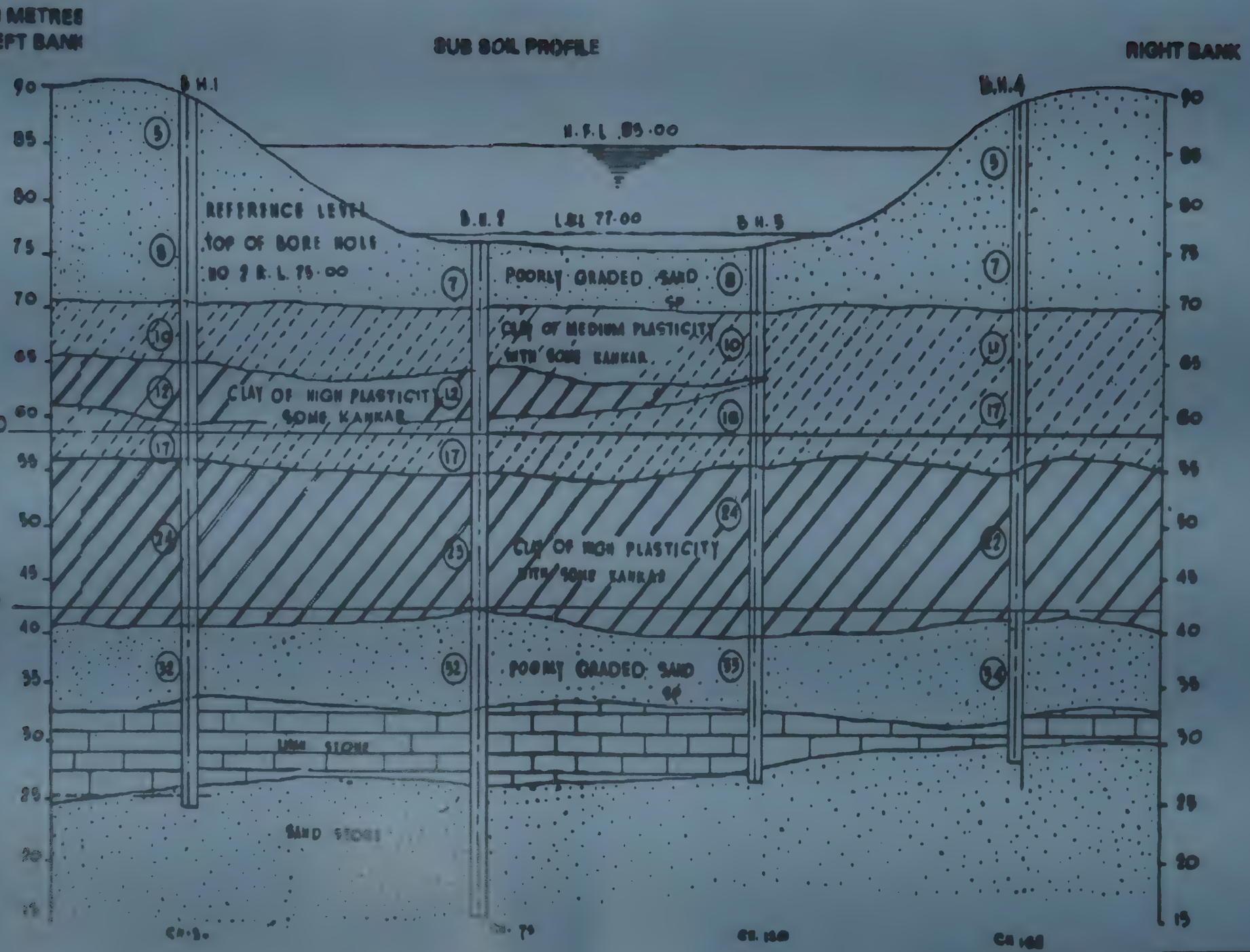


PROJECT

JOB NO.

BORE LOG CHART AND SHEET

BORE HOLE NO.2



Notes :

1. Cross section of River at AA' looking down stream
2. Numbers in the circles represent field S.P.T. n values
3. The details shown are as a sample only

PROJECT———— ——

SUB SOIL PROFILE

CODES AND STANDARDS

All work shall be carried out in accordance with these Technical Specifications, unless otherwise deemed otherwise by the Geotechnical Engineer in writing. Where not specified, the latest edition of the applicable code of practice or procedure as laid down by the Bureau of Indian Standards (BIS) shall be followed. The Earth Manual of the United States Bureau of Reclamation may also be referred to in the absence of relevant information, procedures and guidelines by BIS. Only the SI system shall be observed. Some of the relevant standards issuec by BIS are listed below. However, in case of any discrepancy between these specifications and BIS, these specifications shall govern.

IS : 1080	-	Code of Practice for Design and Construction of Simple Spread Foundations
IS : 1498	-	Classification and Identification of Soil for General Engineering Purpose
IS : 1888	-	Method of Load Tests on Soils
IS : 1892	-	Code of Practice for Site Investigation for Foundations
IS: 1904	-	Code of Practice for Structural Safety of Buildings: Foundations
IS : 2911	-	(Part I to Part IV and all Sections) - Code of Practice for Design and Construction of Piles
IS : 2131	-	Method of Standard Penetration Test for Soils
IS : 2132	-	Code of Practice for Thin-Walled Tube Sampling of Soils
IS : 2470	-	Code of Practice for Installation of Septic Tanks: Part 2 - Secondary Treatment and Disposal of Septic Tank Effluent
IS : 2720	-	Method of Test for Soils (Part I to Part XXXVIII)
IS : 2809	-	Glossary of Terms and Symbols Relating to Soil Engineering
IS : 2810	-	Glossary of Terms and Symbols Relating to Soil Dynamics
IS : 2950	-	Code of Practice for Design and Construction of Raft Foundations
IS : 2974	-	Code of Practice for Design and Construction of Machine Foundations (Part I to Part V)
IS : 4078	-	Code of Practice for Indexing and Storage of Drill Cores
IS : 4091	-	Codes of Practice for Design and Construction of Foundations for Transmission Line Towers and Poles
IS : 4332	-	Methods of Test for Stabilized Soils (Part I to Part X)
IS : 4434	-	Code of Practice for In-Situ Vane Shear Test for Soils
IS : 4464	-	Code of Practice for Presentation of Drilling information and Core Description in Foundation Investigation
IS : 4968	-	Method of Sub-surface Sounding (Part I to Part III)

IS : 5249	-	Method of Test for Determination of In-situ Dynamic Properties of Soils
IS : 5529	-	Code of Practice for In-situ Permeability (Part I and Part II)
IS : 6403	-	Code of Practice for Determination of Allowable Bearing Pressure on Shallow Foundations
IS : 8009	-	Code of Practice for Calculation of Settlements of Foundations (Part I-1976 Part II-1980)
IS : 9214	-	Method of Determination of Modulus of Subgrade Reaction (K value) of Soils in Field
IS : 9221	-	Method for Determination of Modulus of Elasticity and Poisson's Ratio of Rocks Materials in Uniaxial Compression
IS : 10042	-	Code of Practice for Site Investigations for Foundation in Gravel-Boulder Deposit
IS : 10108	-	Code of Practice for Sampling of Soils by Thin Wall Sampler with Stationary Piston
IS : 11550	-	Code of Practice for Field Instrumentation of Swelling Pressure in Expansive Soil
IS : 15376	-	Geological Exploration by Geophysical Method (Electrical Resistivity) – Code of Practice
IRC: SP:123-2017		Guidelines for Geophysical Investigations for Bridges

Appendix-2
(Clause 707.5)

GUIDELINES FOR THE DESIGN AND CONSTRUCTION OF RAFT FOUNDATION FOR ROAD BRIDGES

1. INTRODUCTION

Raft foundations have been successfully used as bridge foundation in most part of India, especially in Maharashtra. It is a common practice to rest the foundations of bridges firmly into sound rock or at a strata of soil below scour level but having adequate safe bearing capacity. Where good founding strata is not available at reasonable depth, deep foundations such as wells, drilled caissons or piles have been adopted. For some of this type of situations an alternative of Raft Foundations can be considered, where construction of Raft slab together with cut off walls to protect the securable strata immediately below the raft is provided. Raft foundations can also be founded on strata of somewhat lower bearing capacities than Rock or Intermediate Geo material.

In case of Raft Foundation, the founding level need not be taken deeper than the maximum scour level like in case of open, isolated footings. It is sufficient to go slightly below general scour depth worked out by LACEY's or other applicable formula. Thus, it reduces the construction problem due to lesser depths of foundations. The full raft and cut off walls along with U/S and D/S stone apron are the barriers to resist washing away of bed material by currents below bed by confinement of underneath strata immediately below the raft slab.

The raft foundations fall under the category of shallow foundations. However, these types of foundations warrant some specific attention pertaining to design, construction and maintenance. For guidance of Bridge Engineers, IRC considered it useful to lay down guidelines for design and construction of Raft foundations. Raft foundation are suitable for small and medium bridges. However, these have been successfully used in Major bridges also. A list of major bridge structures constructed in Maharashtra State using raft foundations with broad design parameters is given in **Annexure-1** for reference.

2. SCOPE

2.1 Scope

The Guidelines can be used for the design and construction of Raft Foundations for bridges on all category of roads. The considerations given in these Guidelines should be supplemented by sound engineering judgment taking in to account the local environment and past performance in the respective regions. For the purpose of these guidelines, Raft Foundation includes Foundation Slabs with structurally detached Cut off walls along with U/S and D/S apron.

2.2 Limitation and acceptability of Raft Foundation

Rivers can be classified into following two categories which decide the suitability of raft foundation hence limitation of these guidelines.

a) Rivers flowing within more or less defined and stable banks consisting of soils like silt, fine to coarse sands.

Raft foundation is a suitable alternative for these rivers; such streams/rivers are typically met in very large part of the country excepting the alluvial belt of Northern and Eastern India and Delta regions of large rivers.

b) Rivers flowing in its own deposited alluvium with large depth of deposits and wide khadir, in which the flood course itself may undergo changes, and even after forming deep local scour-holes the firm bed and sides having cohesive bond between particles are not available. The structure needs guide bunds to control the flood within pre-selected course. Scour depths are several meters deep. Raft type foundations are not suitable for these rivers. However, raft foundations can be considered in this situation also for small streams with limited discharge, medium and small streams of water.

Raft type foundations, if proposed for such conditions, need extensive hydraulic design. For these refer IS: 6966(Part 1), Hydraulic Design of Barrages and Weirs. Also refer to the local experience and practices of the Irrigation Departments dealing with these areas. Certain clarifications for applying these methods to the design of raft foundations are given subsequently.

2.3 Raft in Bouldery strata- Raft foundations should not be constructed in the Bouldery strata or in hilly region where velocity of the flow can be very high.

3. Design and Specifications

3.1 Definition

Raft is a shallow foundation structure consisting of structural slab to support load, cutoff walls at U/S and D/S to restrict the scour along with U/S and D/S stone apron and thereby ensure safety of the structure.

3.2. Elements of Raft Foundations

Reinforced Concrete raft slab is a structural member which transmits the loads to the soil, which is a semi-elastic material. Under the permanent loads the soil settles, mostly unequally, causing permanent settlements and induces corresponding stresses in the raft. Under transient loads the soil acts as an elastic material, causing recoverable stresses and deflections. If soil does not respond so, the raft type foundation is not viable.

The elements of the Raft Foundations, in their most generalised form are shown in **Fig. 1**. These are listed below:

a) Treated and compacted soil below the raft.

b) RCC Raft with thickness designed as per structural and durability requirements, but having concrete of at least M 30 grade. A sacrificial surface layer of not less than 100 mm thickness with surface reinforcement or stone masonry is to be provided where rolling debris are expected during floods. This provision should be extended around the pier for the depth till the velocity of current drops below gravel/pebbles carrying velocity.

c) Vertical Cut off walls on U/S and D/S side of the raft, having water tight connection with the raft but structurally not connected with the same.

d) Stone Apron on upstream with length and thickness designed to make it function as a launching apron to avoid U/S scour immediately in front of the cut off wall and achieve uniform flow from unprotected bed scoured to its deepest normal scour depth to the structure on the U/S. The apron may be provided for short lengths on either side to keep the scour hole away at safer distance.

e) Stone apron on D/S of Raft, with length and thickness designed to make it function as a launching apron so as to avoid D/S scour immediately in front of the cut off (and or concrete blocks) and achieve uniform flow from protection to the scoured bed to its deepest normal scour depth on the D/S.

f) Floor Protection Work-The effective waterway between piers should preferably be adequate to avoid supercritical flow between the piers and a hydraulic jump downstream. However under certain conditions this may not be possible to avoid jump formation. In such situation an extended floor protection consisting of concrete blocks should be provided on D/S immediately after the RCC raft and Cut-off wall. The length is to be decided upon requirement to re-establish subcritical flow as much stream lined as possible and contain formation of hydraulic jump within its length.

g) Toe Wall: Toe wall shall be constructed at the end of Launching Apron

h) Super and Sub Critical Flow-

The dimensionless Froude number, F_r, describes different flow regimes of open channel flow. The Froude number is a ratio of inertial and gravitational forces.

$F_r = V/ (gD)\ 1/2$

Where:

V = Water velocity

D = Hydraulic depth (cross sectional area of flow / top width) g = Acceleration due to Gravity When:

$F_r = 1$, critical flow,

$F_r > 1$, supercritical flow (fast rapid flow),

$F_r < 1$, subcritical flow (slow / tranquil flow)

3.3 Design Considerations for Raft Foundations in Non- Alluvial Rivers

3.3.1 General Consideration

The foundation shall be designed to withstand the worse combination of loads and forces in accordance with the provisions of Clause 706 of IRC: 78. The foundation shall be taken to such a depth that they are safe against Normal scour or protected against it. Apart from this, the depth should be also sufficient from consideration of bearing capacity, settlement, liquefaction potential, stability, etc. at founding level. In case of bridges where the mean scour depth 'd_m' is calculated as per IRC: 5, depth of foundation shall not be less than those of existing structures in the vicinity, which is close enough to influence the flow depth and pattern if the structure is also on Raft.

Although Raft Foundation falls in the category of shallow foundation and is found to be convenient and economical proposition under certain conditions, the general concepts given above need to be interpreted separately, since the foundation itself, acting together with other elements also acts as a bed protection. The requirements of the minimum depth are different from those for open foundations.

3.3.2 Hydraulic and Geotechnical Considerations

The design flood should of not less than 100 years return period as calculated by rational methods for small catchments, but not less than the observed maximum flood and by unit hydrograph method for medium catchments. The requirements of the linear waterway, cross section, constriction, etc., should be worked out on the basis of Manning or Inglis method. The waterway should be calculated as per IRC:5 "Standard Specifications and Code of Practice for Road Bridges, Section I-General Features of Design (Eighth Revision)"

The bearing capacity of the soil should be adequate. The founding strata should be not less than 300 mm below the lowest bed level at low water level. The soil profile of the channel where Raft is to be laid should be more or less uniform so as to provide uniform sub-grade reaction from the soil. One meter depth of soil below this level should be removed and replaced by selected earth, preferably granular material as explained under 4.1.1 to ensure uniform reaction. For initial design appropriate value of sub grade reaction of site soil be based/adopted on Clause 2.3 of IS:9214:1979/Clause 6.4 of IRC:15 which should later be validated by Plate load test. In case of major variation the thickness etc. shall be redesigned. The top of the floor shall be kept atleast 300 mm below lowest bed level to prevent floor acting as a weir when retrogression of levels takes place.

3.3.3 Structural Design of Raft

i) Analysis

Raft slab can be designed as beam on elastic foundation using Heteny's equations for Beams on elastic foundations or as "Plate" resting on elastic support using finite element method or "Grid" with elastic support at every node, loaded at regular interval with dead and live load at pier locations. Any standard software or design charts can be used for this purpose. The raft slab will have structurally detached water tight cut-off wall. The design of structurally integral cut-off wall and the raft is complicated, and the raft may not qualify as a raft slab. The designs done beyond the range of past experience or for extrapolated situations may be verified by model testing and appropriate studies. The average load on the founding strata as a whole shall be within the bearing capacity of soil.

ii) Modulus of Subgrade Reaction (K value)

For selecting appropriate k values IRC:15 / IS: 9214 publications / any text book on soil engineering can be referred for guidance. Confirmatory plate load test can be carried out, if required. Typical k values, normally adopted in the design are reproduced as below:

a) Stiff / hard clay	20 to 25 MPa/m
b) Fine sand	40 to 45 MPa/m
c) Murum or compact sand	50 to 75 MPa/m

iii) Structural Design

Structural design should be in conformity with the applicable Code but thickness not less than 300 mm and of concrete having strength not less than M30. The durability specifications should be for severe conditions of exposures and the provisions of the minimum cover and allowable max crack widths should be suitable for these exposure conditions. The raft slab should be checked for punching shear caused by the concentrated pier load, in addition to the normal bending and shear design for severe conditions of exposure as per **Table 14.1** of IRC:112 subject to verification of local conditions on a particular site. The structural slab should extend beyond pier by at least the amount equal to the design thickness of the slab or equal to width of pier whichever higher.

In addition to the structural thickness, protective measures against abrasion from suspended and rolling coarse material in the flood should be provided. The slab should be laid on 100 mm thick M 10 PCC.

The thickness of the slab should not be less than 300 mm.

iv) Extension of structural slab as floor protection beyond pier/structural thickness

The floor protection in front of piers should be provided for length not less than the width of the pier or depth of raft slab whichever is higher to control/eliminate horse-shoe shaped vortex which causes local scour. The RCC slab should be at least 300 mm thick laid on 100 mm thick M 10 PCC. It should have minimum 0.2% steel of Fe500 grade at top and bottom in each direction. Protective measures against abrasion, if provided, should be continued over the full width.

v) Cut-Off Walls on U/S and D/S of RCC slab

The cut off wall could be attached with raft slab and should be not less than 300 mm thick with M 30 grade concrete. It should be designed as nominally reinforced PCC wall with 0.12% steel of Fe500 grade in each direction on both faces, with clear cover of 75 mm. These walls should not have structural continuity or moment connection with RCC slab, but the joint should be leak tight. The cut off wall should be taken at least 300 mm below the mean depth of scour calculated as per IRC:5.However, the total minimum depth below floor shall be not less than 2 m below the top of floor. In case the substratum with continuous layer of stiff non- securable clay in the neighbourhood of downstream of cut-offs a judicious adjustment in the depths of U/S and D/S cut-offs shall be made to avoid building up pressure under the raft (refer Clause 15.1.1 of IS: 6966).

vi) Pervious Block Protection beyond RCC Raft on D/S side

For cases where hydraulic jump can occur due to presence of supercritical flows as mentioned in Para 3.2. (h) a pervious protection layer consisting of concrete blocks laid on the inverted filter should be provided, for a length needed to achieve desired velocity. The length of the concrete block apron should be at least 1.5 x (depth of average scour below floor level). The minimum size of the concrete blocks should be based on the mean design velocity V in m/s, equivalent diameter in m and weight in kg as given by **Table 1**. For the concrete blocks size should be worked out from weight at density of 2400 kg/cu.m.

Table 1: Size and Weight of Stone for Launching Apron

S. No.	Mean Velocity in m/ Sec-Upto	Maximum Size and Weight of Stone	
		Diameter(mm)	Weight (kg)
1	2.5	300	40
2	3.0	380	76
3	3.5	510	184
4	4.0	670	417
5	4.5	850	852
6	5.5	1040	1561

vii) Launching Aprons on U/S and D/S sides

The length of apron for raft beyond RCC floor slab on U/S side should be at least 1.5 x (depth of average scour on U/S bellow floor level). On D/S side it should extend beyond pervious block (if provided) for at least 2.0 x (depth of average scour on D/S below floor level). However, in no case these shall be less than 3m and 5m respectively.

The thickness of apron shall be 0.6m for Manning's velocity up to 3m/sec at raft top and for velocity more than 3m/sec, the same shall be 1.0 m. However, the thickness of apron shall be such that total quantity of rubble will cover 1:2 sloping length between raft top and normal scour level with 2 layers of 220mm. In case of bed having very fine sand or silt, slope up to 1:3 and in case of boulder reaches 1:1.5 slope may be adopted.

The thickness of loose stone shall be 1.25 times the thickness specified for river slope and type of bed material in IS: 6966. These shall however, have a minimum thickness of 220mm after launching Normally box returns are provided to the bridges with Raft Foundations. In that case, it will be desirable to extend the u/s and d/s stone aprons with Toe walls sideways at least by 3m beyond ends of both abutments on upstream and downstream or anchor them well into stable banks of the stream, to prevent undermining of aprons from sides and to ensure additional safety to box returns and to ensure that box returns do not get undermined during floods.

Size and weight of stone shall be as per **Table 1**. The minimum weight of stone of size D50 shall be 40 kg. Concrete precast or cast in situ block or wire mesh crates can be used to get the required weight. For wire mesh crates refer Para 5.3.7.3, **Appendix-2** of IRC:89.

Where similar Aprons have been provided in the nearby locations and in comparable conditions, the local experience shall be taken in to account.

viii) For a new bridge on raft foundation close to bridge on open or deep foundation closer to less than flood depth following precautions need to be taken:

a) When new raft bridge at U/S-The D/S apron may be extended up to 2m beyond pier base of the existing bridge

b) When new raft bridge at D/S- The U/S apron be extended beyond two metre of existing piers

c) Hydraulics and structural safety of exiting bridge to be rechecked assuming apron top as bed level

In case the U/S or D/S face of the pier of the existing bridge is at least at a distance of 1.5 times the flood depth from the nearest Toe wall of the Apron, then no additional measures, as stated above, are required to be taken.

3.4 Raft Foundation on Alluvial soils

The basic design principles of raft design are the same as above in this case also. The foundation strata below raft is protected from the local extra scour and used as founding strata. The hydraulic design is more complex requiring control of flow on both above and below bed level. Guidelines for doing so are available in IS:6966(Part 1), "Hydraulic Design of Barrages and Weirs".

While using the same, apart from non-applicability of weir/barrage related clauses, certain other basic differences have to be kept in mind. These are described below:

i) The bridge sub structure does not impound any reservoir and the difference between U/S and D/S levels of the water surface are due to natural longitudinal gradient in open channel, local change to the shear resistance by the raft and protective aprons and the afflux created by the obstruction of piers resulting in reduced waterway and increased local velocities. This is comparatively much smaller as compared to the weir/barrage. Hence the hydraulic design of flow below bed is easier to manage. The requirements given in the IS can be reduced/ relaxed as mentioned below. ii) Clause 4: Data requirements can be relaxed in their extent and coverage iii) Clause 5: Design flood of 500 years return period need not be used.

iv) Clause 6: The afflux restriction of 0.5 m and 1.0 m are not applicable.

v) Clause 14: Energy dissipation may or may not be required, depending upon nature and energy of flow between the piers and the flow on the D/S beyond the structure and further down in the original sections of the river. This may occur by formation of hydraulic jump or by creation of obstacles to the flow and resulting turbulent flow. It is necessary to provide D/S RCC slab and pervious concrete blocks of adequate length to ensure that the jump is contained within this portion.

The Launching apron has to be provided beyond the Concrete block protection.

4. Construction Requirements

4.1 Foundation Preparation

It is desirable to carry out construction of raft in dry season. Raft foundations are constructed by excavating entire bed up to the depth of cut off walls. The form work is then erected and reinforcement for Cut off placed in place. Sometimes, it is not possible to excavate the entire bed due to water seepage. In such situations small portion of cut off wall is excavated for limited width and full depth. The excavated portion is concreted, and work progressed. Appropriate type of concrete should be used to avoid washing out of cement slurry from concrete. Use of dry concrete shall be avoided as its quality control is difficult. The raft is a RCC structural member transferring the load to the soil below. The bearing capacity of the soil supporting the Raft slab should be adequate. The founding strata should be at-least 0.30 m below the lowest bed level at low water level. The soil profile below this level should be more or less uniform so as to provide uniform sub-grade reaction from the soil. In absence of uniformity in the strata, 1m depth of soil below this level should be removed and the bed rolled and compacted using good selected non- cohesive soil. Over this compacted bed selected earth from suitable borrow pits should be backfilled in layers and compacted to 95% proctor density. A PCC layer of about 100 mm thickness shall be laid over the compacted earth. A layer of about 100mm shall be constructed in such a manner that it does not get mixed with the soil. It provides a solid base for the raft foundation and a mix of M 10 concrete can be used in it. Plain concrete is vibrated to

achieve full compaction. Concrete placed below ground should be protected from falling earth during and after placing. Concrete placed in ground containing deleterious substances should be kept free from contact with such a ground and with water draining it during placing and for a period of seven days. When joint in a layer of concrete are unavoidable, and end is sloped at an angle of 30° and junctions of different layers break joint in laying upper layer of concrete. The lower surface is made rough and clean watered before upper layer is laid. Raft slab of required thickness shall be laid over the compacted PCC layer. Substructure should not be started unless raft slab along with Apron is completed.

4.2 Cut Off walls

Bed shall be excavated to construct Cut-off walls abutting to the Raft Slab on U/S and D/S sides of the Raft. The Cut off walls can be integrated to the Raft slab to have a water leak-proof joint. The joint of Cut-off walls and Raft slab shall be watertight and the same can be achieved if required by providing PVC water-stop with minimum embedment of 150 mm. However, the Cut off walls should not share moments from the raft slab as these are not designed for that. The Cut-off walls shall be minimum 300 mm thick in M 30 Concrete with minimum depth as 2.0 m or 300 mm below the mean scour depth whichever is more. The construction of Raft and Cut off walls shall be taken up during dry season to avoid dewatering problem. In case of excessive water percolation during construction of Cut-off walls sinking of Precast Cut off walls, dumbbell shape "well" with U/S and D/S cut-off connected together to be sunk by grabbing, or sheet pile techniques or bentonite clay technique or similar such construction techniques be adopted. Cross cutoff wall of 2.3 m depth shall be provided at the end of raft so as to prevent washing out of the filling beneath the Raft. In case full length of raft cannot not be completed before the onset of monsoon then in that case also the cross cutoff shall be provided at end of constructed portion of the raft. These will strengthen the main cut off walls as well as protect the underneath bed from scouring in case of out-flanking

4.3 Protective Block Protection on D/S side

As explained in Para 3.2. (f)additional protective block protection at D/S need to be provided in case supercritical velocity is likely to be generated.

4.4 Launching Apron on U/S and D/S sides

Launching apron at U/S and D/S are essential element of the system and detailed requirement are defined in Para 3.2 (g).

4.5 Special Cases

Construction of Raft Slab and cut off walls up to desired depth by excavating an open trench is possible in cohesive soils. However, in case of sandy soils and structures near irrigated areas dewatering is one of the major difficulties faced in the construction of raft foundations. Various innovative methods have been devised and put into practice in the field to overcome such difficulties. Also, Raft need not be constructed for the entire width of superstructure. Narrow Raft with proper flare has also been designed and constructed in past. It is not necessary that the width of raft slab is same as that of superstructure. Raft slab width can be varied to achieve economy. Width of central portion can be reduced while at end it can be matched with abutment width. Typical sketch is given in **Fig. 2**.

4.6 Combination of Foundations

Sometimes rock may be available at surface or in shallow depth only in part width of the river. Rock if available at isolated small location then same can be removed at least for 1m below the bottom of raft slab and filled up with selected granular material to be well compacted. Where the rock is available in part length for full width of raft, in such cases the system of open foundation in rocky portion and raft for remaining length can be adopted. However, in such case the provision of cross cut-off at the end of raft portion need to be given. In addition to this the span next to raft foundation need to be fully protected against scour by U/S and D/S cut-off, nominal concrete raft slab as bed protection along with U/S and D/S apron as bed protection .

5 Precautions to be taken

The following precautions shall be taken during design, construction and maintenance.

5.1 Preparation of Bed

The excavated area below the raft slab should be well consolidated and if required necessary treatment should be applied to soil to achieve the desired bearing capacity depending on soil investigation data.

5.2 Control on exit gradient and Bed Protection

The soil below the Raft slab needs to be protected from erosion. The scour of the soil immediately around the substructure is protected by providing cut off walls abutting the raft slab. The scour on the upstream or downstream side beyond the cut off walls may take place where bed protection is not provided. Launching stone aprons should be provided for adequate length on U/S and D/S so that the scour holes do not extend below the foundation level. The Raft, flooring and cutoff walls length should be such that the exit gradient and exit velocity do not exceed the permissible limits.

5.3 Uniform Soil reaction to Raft slab

The Raft slab is designed as supported for the entire length and width by well compacted natural granular or with selected material. Such a bed provides vertical reaction to the RCC Raft slab when loaded.The bedding material should be of same type.

5.4 Maintenance of Raft Foundation

The satisfactory and safe performance of raft foundations depends on the function of Raft slab, Cut off, elastic bed material below the slab, launching apron and Toe walls. It is, therefore necessary to ensure that all these elements remain intact. These elements should be inspected at least twice a year before and after monsoon. Any deficiency found should be made good well before the onset of monsoon.

Top of the Raft slab in part stretch shall be cleared and closely inspected. If any damages are noticed then the whole Raft slab shall be cleared, inspected and repaired if necessary.

U/S and D/S launching Apron shall be inspected and dislodgement of stones shall be replaced with appropriate quantity of stones of desired weight. Similarly, toe walls shall also be inspected and repaired if needed.

Scouring of bed material below Raft slab needs to be monitored and protected. Hollows below the raft slab with or without appearance are not permitted as it may lead to settlement and cracking of Raft slab.

5.5 Sand Dredging

Dredging of sand at least for a length of 6 times the river width at U/S and D/S or minimum 300 m, whichever is more should not be allowed. This zone should be declared a prohibited zone and no digging or the use of area within the zone should be allowed.

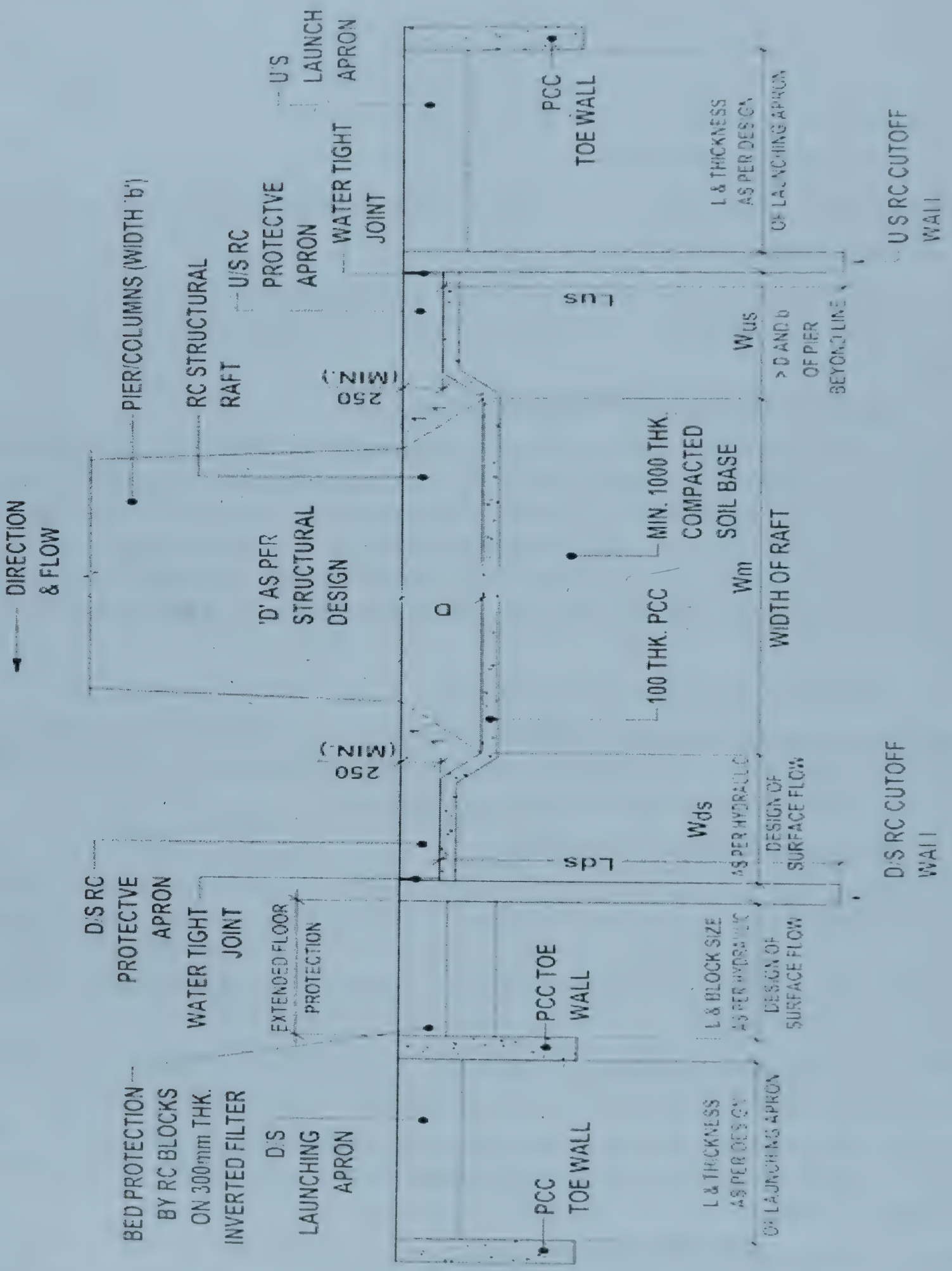


Fig. 1 Typical Cross Section of Raft Foundation

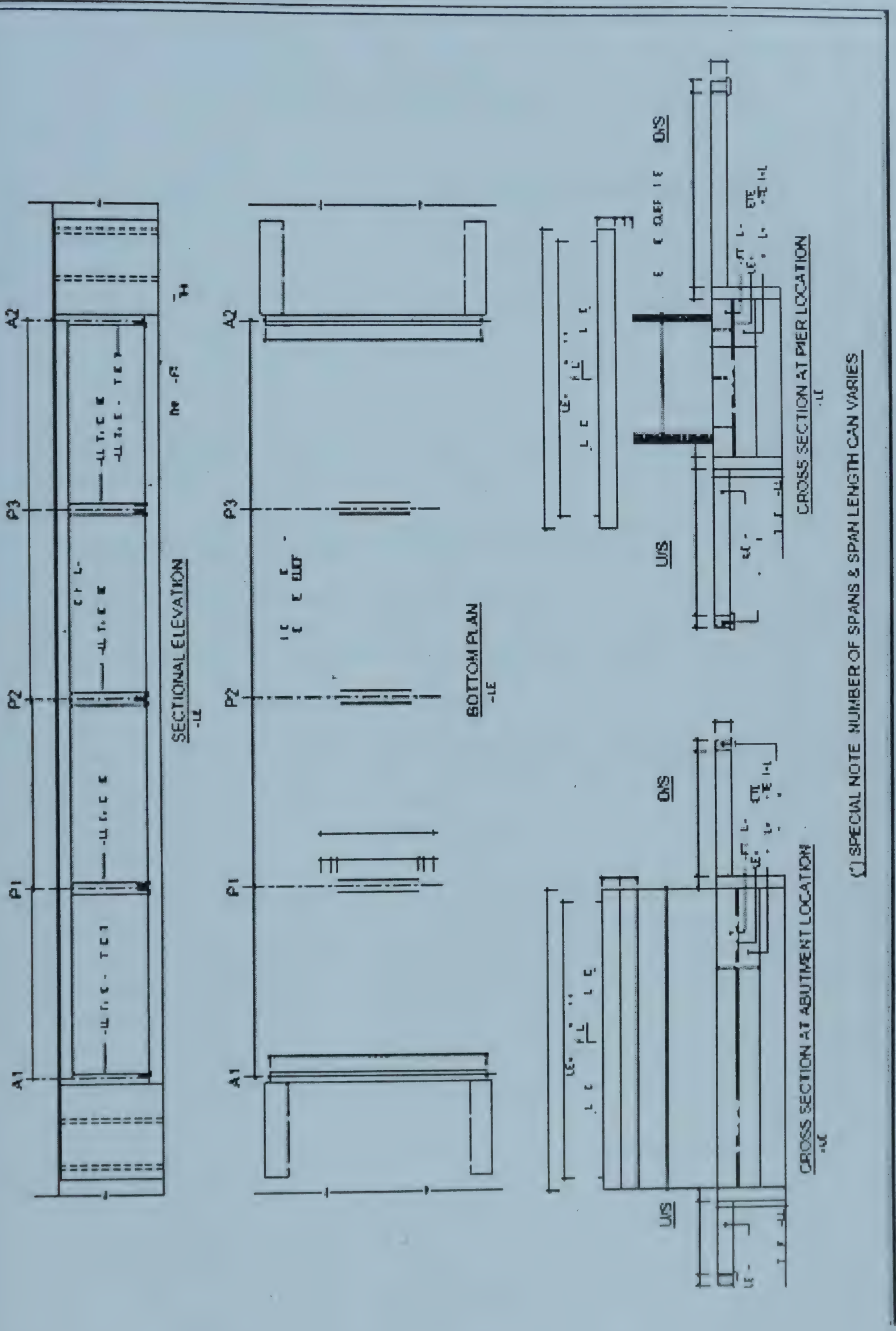


Fig. 2 Typical Details of Raft showing reduction in width

List of Major Bridges with Raft Foundation built in Maharashtra State

Sr. no	Name of road and standard NH/SH/ MDR/ODR/ VR	Name of Stream	Carria-geway width in m	Total length of bridge in m	Spanning arran-gement	Pier Dimension in plan.	Soil type clayey, sandy etc	Design discharge	Mean Design HFL	Design scour level.	Depth of flow	Grade of Concrete	Thickness of Raft slab	Type Attached / Detached	Thickness	Height of cut off wall	Length upstream apron	Length of downstream apron	Thickness of apron
1	Bhandara-Tumsar-Balaghat Rd. (SH)	Bawanthadi	7.5	423	47 Spans of 9.0 m. c/c	9.02 x 1.52	Sandy	6157.857	263.8	253.39	6.16	M 20	0.54	Det	0.35	4.25	12	15	1.2
2	Itkheda-Butai-Keshori Rd (MDR)	Gadhavi	7.5	168	28 Spans of 6.0 m. c/c	8.88 x 1.39	Sandy	2696.25	99.038	91.236	5.502	M 30	0.375	Det	0.4	2.3	4	6	0.55
3	Jamgaon-Thadipaoni Rd(MDR)	Wardha	7.5	130	13 Spans of 10.0 m. c/c	9.30 x 1.803	Sandy	5243	96.41	83.085	11	M 30	0.65	Det	0.4	2.3	6	8	0.6
4	Anjangaon-Akot Rd (SH)	Shahanur	9	120	15 Spans of 8 m. c/c	10.472 x 1.472	Silty	5026	102.41	89.837	8.573	M 30	0.5	Det	0.5	4	6	9	0.6
5	Rajoli-Approach Rd (VR)	Kathani	4.25	114	19 Spans of 6.0 m. c/c	5.58 x 1.33	Sandy	2674.649	101.17	86.244	13	M 30	0.55	Det	0.4	2.3	4	6	0.65
6	Akola-Akot Rd (SH)	Purna	7.5	96	8 Spans of 12.0 m. c/c	8.50 X 2.0	Silty	10909	109.54	88.6	18	M 30	0.4	Det	0.5	3	13.45	21.75	1

Appendix-3

(Clauses 708.2, 708.3 and 708.4)

PROCEDURE FOR STABILITY CALCULATION

1. FORMULA FOR ACTIVE OR PASSSIVE PRESSURE IN SOIL

The active and passive pressure co-efficient (K_a & K_p respectively) shall be calculated according to Coulomb's formula taking into account the wall friction. For cohesive soils, the effect of 'c' may be added to the same as per procedure given by Bell. The value of angle of wall friction may be taken as 2/3rd of ø the angle of repose is subject to a limit of 22½ degrees. Both the vertical and horizontal components shall be considered in the stability calculations.

2 SKIN FRICTION

The relief due to skin friction shall be ignored However, in case of highly compressive soils, skin friction, if any, may cause increased bearing pressure on the foundation and shall be duly considered.

3 FACTOR OF SAFETY OVER ULTIMATE PRESSURES

The factor of safety in assessing the allowable passive resistance shall be 2 for load combinations without wind or seismic forces and 1.6 for load combinations with wind or seismic forces. The manner of applying factor of safety shall be as indicated below:

i) Pier wells

The factor of safety as stipulated for the type of soil shall be applied for the net ultimate soil resistance viz. ($P_p - P_a$) where $P_p - P_a$ are total passive and active pressure respectively mobilized below the maximum scour level.

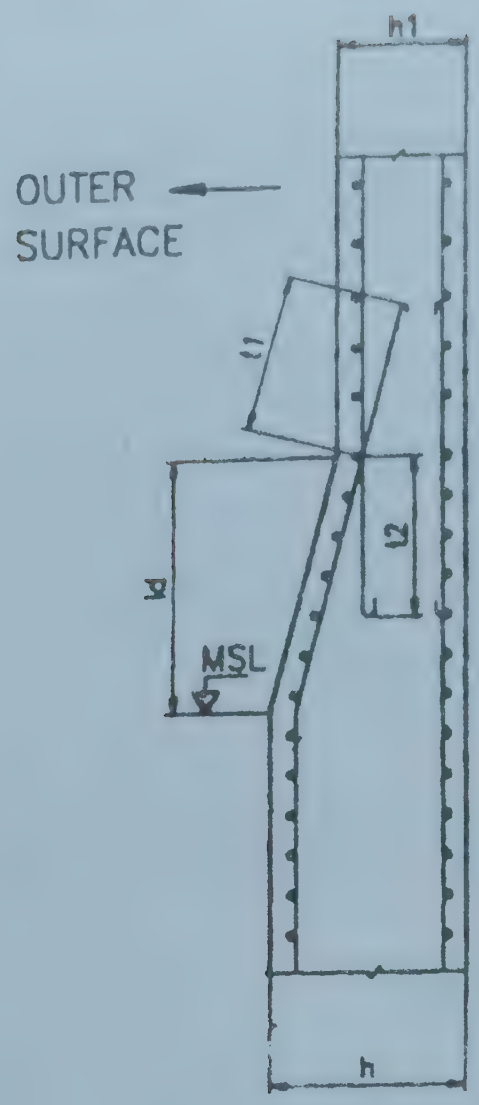
ii) Abutment wells

In the case of abutment wells, the active pressure on soil above the maximum scour level (tri angular variation of pressure) shall be separately evaluated and considered as load combined with the other loads acting on the abutment and no factor of safety sha!! be taken for the above components of active pressure. Effects of surcharge due to live load should be restricted only up to the abutment portion.

iii) However, the lateral resistance of soil below the scour level at ultimate value shall be divided by the appropriate factor of safety, viz., $\frac{(P - P_a)}{FOS}$

iv) Point of rotation

For the purpose of applying the above formulae, it may be assumed that the point of rotation lies at the bottom of the well.



$h = Kd \sqrt{1}$

$h1 = Kd1 \sqrt{1s}$

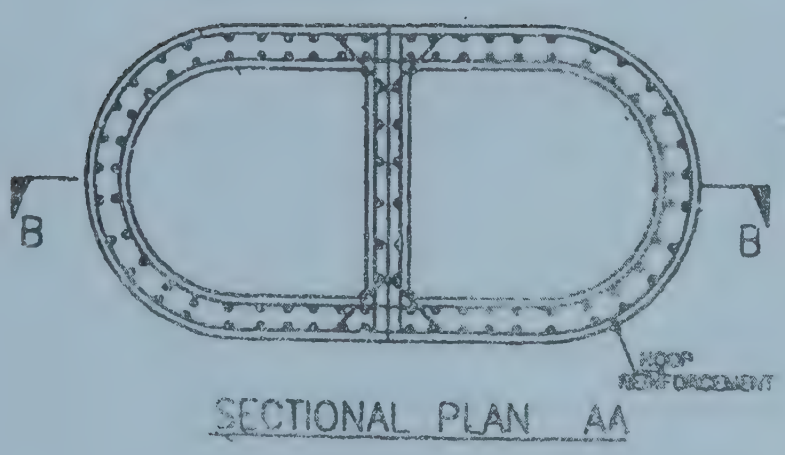
WHERE d1 = OUTER DIA OF WELL AFTER REDUCTION IN STEINING THICKNESS

IS = DEPTH OF WELL UPTO MSL

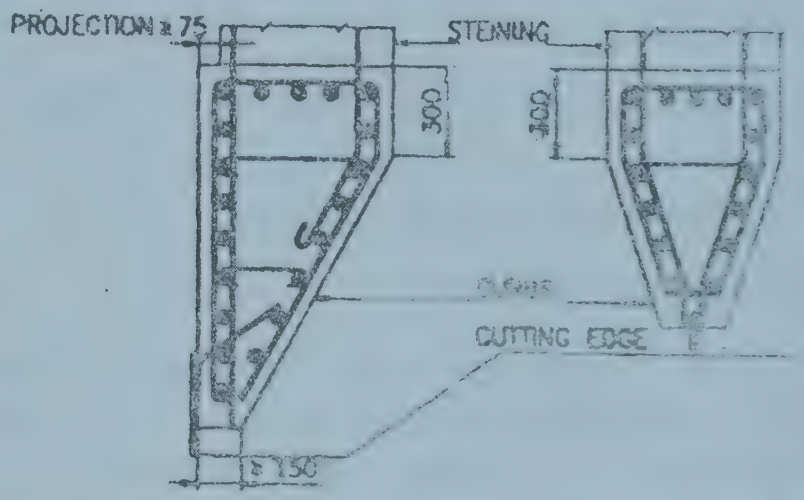
$ld = 3 (h-h1)$

t1 &, t2 ARE THE DEVELOPMENT LENGTHS FOR THE STEEL. BEYOND THE MINIMUM SECTION

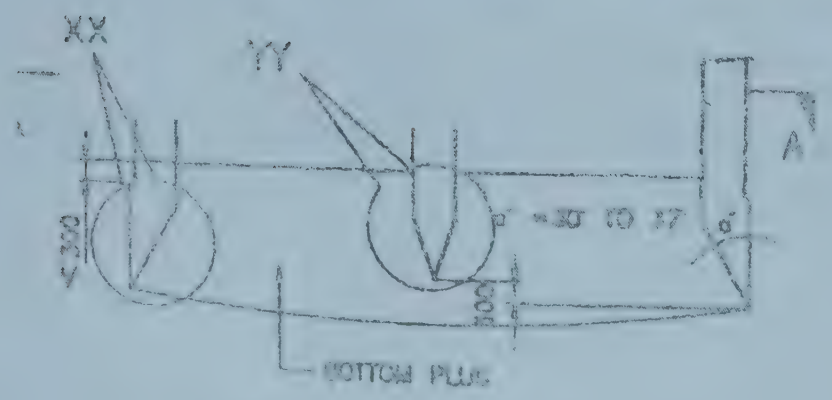
Fig. 1 Sketch for Reduction of Steining Thickness



SECTIONAL PLAN AA



DETAIL AT XX



SECTIONAL ELEVATION BB
(REINFORCEMENT NOT SHOWN)

Fig. 2 Details of Well Curbs and Cutting Edge

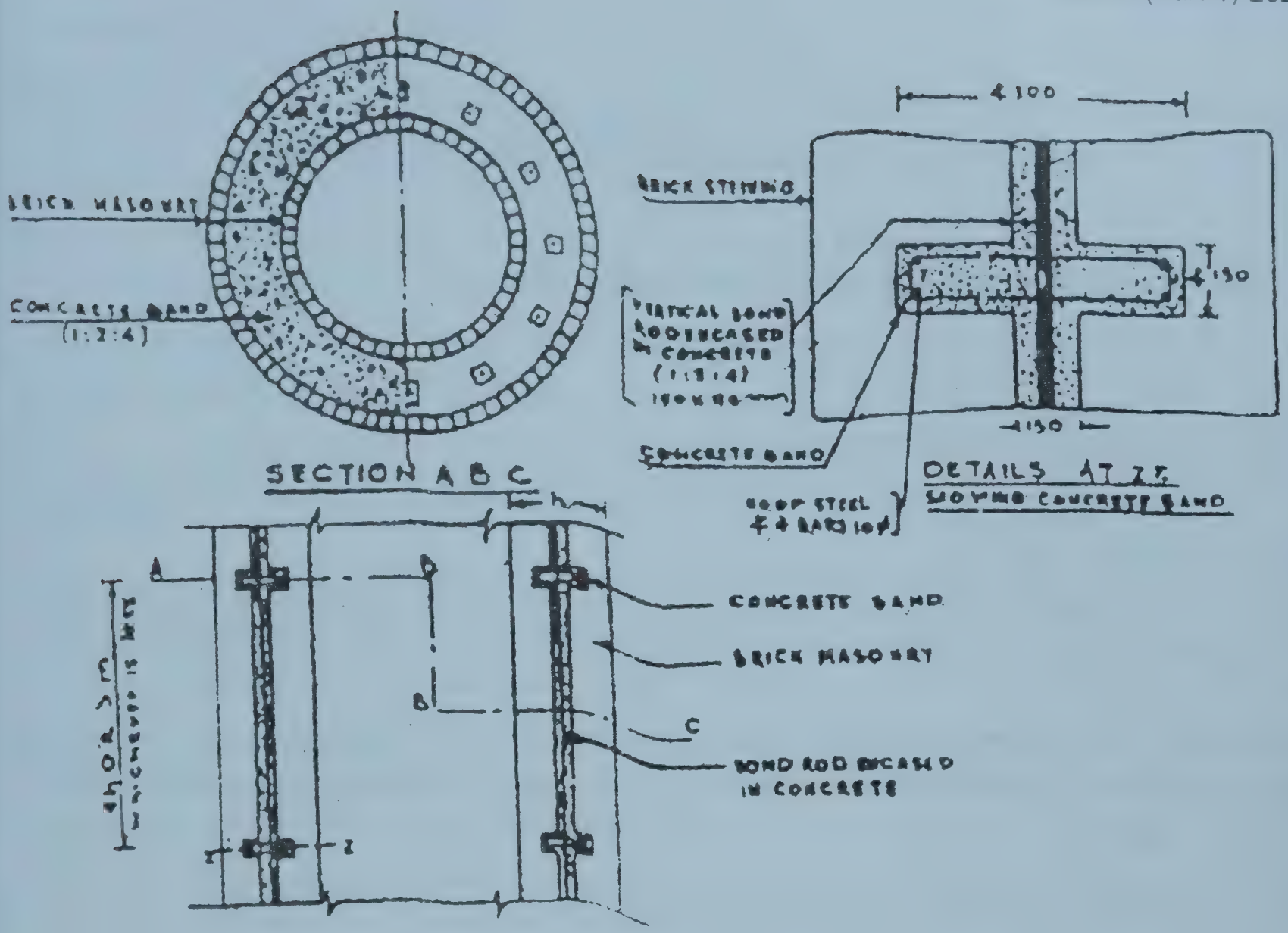


Fig. 3 Details of Well Curbs and Cutting Edge

Appendix-4

(Clause 708.13)

PRECAUTIONS TO BE TAKEN DURING SINKING OF WELLS

1. CONSTRUCTION OF WELL CURB AND STEINING

1.1 Cutting edge and the top of the well curb shall be placed truly horizontal.

1.2 The methods adopted by placing of the wellcurb shall depend on the site conditions. and the cutting edge shall be placed on dry bed.

1.3 Well steining shall be built in lifts and the first lift shall be la'd after sinking the curb atleast partially for stability.

1.4 The steining shall be built in one straight line from bottom to top and shall always be at right angle to the plane of the curb. In no case, it shall be built plumb in intermediate stages when the well is tilted.

In soft strata prone to settlement/creep, the construction of the abutment wells shall be taken up after the approach embankment for a sufficient distance near the abutment has been completed.

2. SINKING

2.1 A sinking history record be maintained at site.

2.2 Efforts shall be made to sink wells true to position and in plumb.

2.3 Sumps made by dredging below cutting edge shall preferab'y not be more than half the internal diameter.

2.4 Boring chart shall be referred to constantly during sinking for taking adequate care while piercing different types of strata by keeping the boring chart at the site and plotting the soil as obtained for the well steining and comparing it with earlier bore data to take prompt decisions.

2.5 When the wells have to be sunk close to each other and the clear distance is less than the diameter of the wells, they shall normally be sunk in such a manner that the difference in the levels of sump and the cutting edge in the two wells do not exceed half the clear gap between them.

2.6 When group of wells are near each other, special care is needed to ensure that they do not fall in the course of sinking and also do not cause disturbance to wells already sunk. The minimum clearance between the wells shall be half the external diameter. Simultaneous and level dredging shall be carried out in the dredging holes of all the wells in the group and plugging of all the wells be done together.

2.7 During construction partially sunk wells shall be taken to a safe depth below the anticipated scour levels to ensure their safety during ensuing floods.

2.8 Dredging material shall not be deposited unevenly around the well.

3. USE OF KENTLEDGE

3.1 Where a well is loaded with Kentledge to provide additional sinking effort, such load shall be placed evenly on the loading platform, leaving sufficient space in the middle to remove excavated material.

3.2 Where tilts are present or there is a danger of well developing tilt, the position of the load shall be regulated in such a manner as to provide greater sinking effort on the higher side of the well.

4. SAND BLOWS IN WELLS

4.1 Dewatering shall be avoided if sand blows are expected. Any equipment and men working inside the well shall be brought out of the well as soon as there are any indications of a sand-blow

4.2 Sand blowing in wells can often be minimized by keeping the level of water inside the well higher than the water table and also by adding heavy kentledge.

5. SINKING OF WELLS WITH SE OF DIVERS

5.1 Use of divers may be made in well sinking both for sinking purposes, like, removal of obstructions, rock blasting, etc. as also for inspection. All safety precautions shall be taken as per any acceptable safety code for sinking with divers or any salutary regulations in force.

5.2 Only persons trained for the diving operation shall be employed. They shall work under expert supervision. The diving and other equipments shall be of an acceptable standard. It shall be well maintained for safe use.

5.3 Arrangement for ample supply of low pressure clean cool air shall be ensured through an armored flexible hose pipe. Standby compressor plant will have to be provided in case of breakdown.

5.4 Separate high pressure connection for use of pneumatic tools shall be made. Electric lights, where provided, shall be at 50 volts (maximum). The raising of the diver from the bottom of wells shall be controlled so that the decompression rate for divers conforms to the appropriate rate as laid down in the regulation.

5.5 All men employed for diving purpose shall be certified to be fit for diving by an approved doctor.

6. BLASTING

6.1 Only light charges shall be used under ordinary circumstances and should be fired under water well below the cutting edge so that there is no change of the curb being damaged.

6.2 There shall be no equipment inside the well nor there any labour in the close vicinity of the well at the time of exploding the charges.

6.3 All safety precautions shall be taken as per IS:4081 "Safety Code for Blasting and Related Drilling Operations" to the extent applicable, whenever blasting is resorted to. Use of large charges, 0.7 kg. or above may not be allowed except under expert direction and with

permission from Engineer-in-Charge. Suitable pattern of charges may be arranged with delay detonators to reduce the number of charges fired at a time. The burden of the charge may be limited to 1 m and the spacing of holes may normally be kept 0.5 to 0.6 m.

6.4 If rock blasting is to be done for seating of the well, the damage caused by the flying debris should be minimized by provisions of rubber mats covered over the blasting holes before blasting.

7. PNEUMATIC SINKING

7.1 The pneumatic sinking plan and other allied machinery shall not only be of proper design and make, but also shall be worked by competent and well trained personnel. Every part of the machinery and its fixtures shall be minutely examined before installation and use. Appropriate spares, standbys, safety of personnel as recommended in the IS:4188 for working in compressed air must be kept at site. Safety code for working in and other labour laws and practices prevalent in the country, as specified to provide safe, efficient and expeditious sinking shall be followed.

7.2 Inflammable materials shall not be taken into air locks and smoking shall be prohibited.

7.3 Whenever gasses are suspected to be oozing out of dredge hole, the same shall be analysed by trained personnel and necessary precautions adopted to avoid hazard to life and equipment.

7.4 Where blasting is resorted to, it shall be carefully controlled and all precautions regarding blasting shall be observed. Workers shall be allowed inside after blasting only when a competent and qualified person has examined the chamber and steining thoroughly.

7.5 The weight of pneumatic platform and that of steining and kentledge, if any, shall be sufficient to resist the uplift from air inside, skin friction being neglected in this case.

7.6 If at any section the total weight acting downwards is less than the uplift pressure of air inside additional kentledge shall be placed on the well.

7.7 If it is possible to make the well heavy enough during excavation, "Blowing Down" may be used. The man should be withdrawn and the air pressure reduced. The well should then begin to move with a small reduction in air pressure. "Blowing Down" should only be used where the ground is such that it will not heave up inside the chamber when the pressure is reduced. When the well does not move with a reduction in air pressure, kentledge should be added. Blowing down should be in short stages and the drop should not exceed 0.5 m of any stage. To control sinking during blowing down, use of packs or packagings may be made.

8. TILTS AND SHIFTS OF CAISONS WELLS

8.1 Tilts and shifts shall be carefully checked and recorded regularly during sinking operations. For the purpose of measuring the tilts along and perpendicular to the axis of the bridge, level marks at regular intervals shall be painted on the surface of the steining of the well.

8.2 Whenever any tilt is noticed, adequate preventive measures, like, putting eccentric Kentledge, pulling strutting, anchoring or dredging unevenly and depositing dredge material unequally, putting obstacles below cutting edge, after jetting etc. shall be adopted before any

further sinking. After correction, the dredged material placed unevenly shall be spread evenly.

8.3 A pair of wells close to each other have a tendency to come closer while sinking. Timber struts may be introduced in between the steining of these wells to prevent tilting.

8.4 Tilts occurring in a well during sinking in dipping rocky strata can be safeguarded by suitably supporting the kerb.

9. SAND ISLAND

9.1 Sand island where provided shall be protected against scour and the top level shall be sufficiently above the prevailing water level so that it is safe against wave action.

9.2 The dimension of the sand island shall not be less than three times the dimension in plan of the well or caisson.

Appendix-5

(Clause 709.3.1)

CAPACITY OF PILE BASED ON PILE SOIL INTERACTION

1. AXIAL CAPACITY OF PILES IN SOIL

Axial load carrying capacity of the pile is initially determined by calculating resistance from end bearing at toe/tip or wall friction/skin friction along pile surface or both. Based on the soil data, the ultimate load carrying capacity (Q_u) is given by:

$Q_u = R_u + R_1$

Where

R_u = Ultimate base resistance

R_f = Ultimate shaft resistance

1.1 R_u = R_u, i.e., Ultimate base resistance may be calculating from the following:

$R_u = A_p (1/2\ DYN_y + P_d N_q) + A_p N_c C_p$

Where

A_p = Cross – section area of base of pile

D = Pile diameter in m

γ = Effective unit weight of soil at pile tip **= (bulk unit weight of soil- unit weight of water) in** kg/cm^3

N_q & N_y = Bearing capacity factors based on angle of internal friction of pile tip

N_c = Bearing capacity factor usually taken as 9

C_p = Average cohesion at pile tip (from unconsolidated **undrained** test)

P_d = Effective overburden pressure at pile tip limited to 20 times diameter of pile for piles having length equal to more than 20 times diameter.

2. R_f i.e. = Ultimate side resistance may be calculated from the following
Where

K = Coefficient of earth pressure

P_{di} = Effective overburden pressure in kg/cm^2 along the embedment of pile for the ith layer, where i varies from I to n

δ = Angle of wall friction between pile and soil in degrees. It may be taken equal to angle of internal friction of so l

A_{si} = Surface area of pile shaft in cm^2 in the i^{th} layer, where i varies from 1 to n

A_s = Surface area of pile shaft in cm^2

α = Reduction factor

C = Average cohesion in kg/cm^2 throughout the embedded length of pile (from unconsolidated **undrained** test)

3 While evaluating effective overburden pressure, total and submerged weight of soil shall be considered above and below water table respectively.

4. The initial value of K may be taken as 1.5 which can be further increased up to 1.8 in particular cases as specified in Clause 709.2.2 (v).

5. The following value of α may be adopted depending upon consistency of soil:

Consistency	N Value	Bored piles Cast-in-situ	Driven case-in-situ piles
Soft to very soft clay	<4	0.7	1
Medium	4-8	0.5	0.7
Stiff	8-15	0.4	0.4
Very stiff	> 15	0.30	0.3

6. For piles in over consolidated soils, the drained capacity may be evaluated.

7. When full static penetration data is available for the entire depth, then

Q_u = $q_b A_p + f_s . A_s$

q_b = Point resistance at base to be taken as average of the value over a depth equal to 3 times the diameter of pile above and one time the diameter of pile below the tip.

A_p = Cross-sectional area of base of pile

f_s = Average side friction and following co-relation may be used as a guide:

Type of Soil Clay	Side Friction, f_s
Soft	q_c /25
Stiff	q_c /15

Mixture of silts and sand with traces of clay	
Loose	q_c /25
Dense	q_c /15

q_c = Static point resistance.

8. Where soft compressible clay layer is encountered, and contribution towards capacity of pile from such soil shall be ignored and additional load pile on account of downward drag on pile due to consolidation of soft soil shall be considered.

Note: For factors of safety of piles in soil, refer Clause 709.3.2.

9 CAPACITY OF PILES IN INTERMEDIATE GEO-MATERIAL AND ROCK

9.1 Axial load carrying capacity:

Piles in rocks and weathered rocks of varying degree of weathering derive their capacity by end bearing and socket side resistance. The ultimate load carrying capacity may be calculated from one of the two approaches given below:

Where cores of the rock can be taken and unconfined compressive strength directly established using standard method of testing, the approach described in method 1 shall be used. In situations where strata is highly fragmented, where RQD is nil or (CR+RQD)/2 is less than 30 percent, or where strata is not classified as a granular or clayey soil, or when the crushing strength is less than 10 MPa, the approach described in method 2 shall be used. Also, for weak rock like chalk, mud stone, clay stone, shale and other intermediate rocks, method 2 is applicable.

Method 1:

$$Q_u = R_e + R_{af} = K_{sp}.q_c\, d_f.\, A_b + A_s\, C_{us}$$

$$Q_{allow} = (R_e/3) + (R_{af}/6)$$

Where,

Q_u = Ultimate capacity of pile socketed into rock in Newtons

Qallow = Allowable capacity of Pile

R_e = Ultimate and bearing

R_{af} = Ultimate side socket shear

K_{sp} = An empiricalco-efficient whose value ranges from 0.3 to 1.20 as per the table below for the rocks where core recovery is reported, and cores tested for uniaxial compressive strength.

(CR + RQD)/2	K_{sp}
30 %	0.3
100 %	1.2

CR = Core Recovery in percent

RQD = Rock Quality Designation in percent

For Intermediate values, K_{sp} shall be linearly interpolated

q_c = Average unconfined compressive strength of rock core below base of pile for the depth twice the diameter/least lateral dimension of pile in MPa

A_b = Cross-sectional area of base of pile

d_f = Depth factor = $1 + 0.4 \times \frac{\text{Length of Socket}}{\text{Diameter of Socket}}$

However, value of d_f should not be taken more than 1.2.

A_s = Surface area of socket

C_{us} = Ultimate shear strength of rock along socket length,

= 0.225 $\sqrt{qc}$, but restricted to shear capacity of concrete of the pile, to be taken as 3.0 MPa for M 35 concrete in confined condition, which for other strength of concrete can be modified by a factor $\sqrt{(f_{ck}/35)}$.

METHOD 2:

This method is applicable when cores and/or core testing result are not available, or when geo-material is highly fragmented. The shear strength of geo-material is obtained from its correlation with extrapolated SPT values for 300 mm of penetration as given in table below:

Shear Strength/Consistency	Moderately Weak	Weak	Very Weak
Approx. N Value	300 -200	200 -100	100 – 60
Shear Strength/ Cohesion in MPa	3.3 -1.9	1.9 - 0.7	0.7 – 0.4

$Q_u = R_e + R_{af} = C_{ub} N_c A_b + C_{us} A_s$

$Q_{allow} = (R_e/3) + (R_{af}/6)$

Where

C_{ub} = Average shear strength below base of pile, for the depth equal to twice the diameter/least lateral dimension of pile, based on average 'N' value of this region

C_{us} = Ultimate shear strength along socket length, to be obtained from table, based on average 'N' value of socket portion. This shall be restricted to shear capacity of concrete of the pile, to be taken as 3.0 MPa for M 35 concrete in confined condition, which for other strengths of concrete can be modified by a factor $\sqrt{(fck/35)}$ Intermediate values c_{ub} and c_{us} can be interpolated linearly.

L = Length of socket.

N_c = 9

Q_{allow} = Allowable capacity of pile.

The extrapolated values of 'N' greater than 300 shall be limited to 300 while using this method.

General notes common to Method 1 and Method 2:

1) For the hinged piles resting on rock proper seating has to be ensured. The minimum socket length should be 300 mm in hard rock, and 0.5 times the diameter of the pile in weathered rock.

2) The allowable end bearing component after dividing by factor of safety shall be restricted to 5 MPa.

3) For calculation of socket friction capacity, the top rock 300 mm depth of rock shall be neglected. The friction capacity shall be further limited to depth of six times diameter of pile.

4) For the termination of working piles in the rocky strata methodology given in sub-clause 10 can be used as a quality control tool.

a) Moment Carrying Capacity of Socketed Piles:

For the socketed pile, the socket length in the rock may be calculated from following equation:

$$L_s = \frac{2H}{\sigma_1 D} + \sqrt{\frac{4H^2}{\sigma_1^2 D^2} + \frac{6M}{\sigma_1 D}}$$

Where,

L_s = Socket length

H = Horizontal force at top of the socket

M = Moment at the top of the socket

D = Diameter of the pile.

σ_1 = Permissible compressive strength in rock which is leaser of 30 kg/sq. cm or $0.33q_c$

In case of socketed piles, for the satisfactory performance of the socket as fixed tip, the rotation at the top of the socket for the fixed condition (9) should be less than or equal to 5 percent of the rotation for the pinned condition at the top of the socket (Θ)

10. PILE TERMINATION CRITERIA AS A QUALITY CONTROL TOOL IN ROCKS

For establishing the similarity of soil strata actually met while advancing the pile-bore with the strata selected for terminating the pile on the basis of N values equivalent energy method can be used.

The concept of Pile Penetration Ratio (PPR) is used in this method.

The pile penetration ratio (PPR) reflects the energy in ton-meter required to advance the pile bore of one sq. metre

Cross-sectional area by 1 cm.

1) In case of SPT test its PPR can be worked out as follows:

Energy E spent for N blows = 63.5 kg x 75 cm x N blows (in kg – cm units) = E x 10^{-5} ton metre.

Area of samples is **0.785 x $(5.08)^2$ sq. m = 20.258 x 10^{-4} sq.m, for penetration of 30 cm.**

PPR = 63.5 x 75 x N x / (20. 258x10^{-4}x 30) =0.784N T.M/M^2/cm

for N = 50 = 39.20 T_m / m^2 / cm

and for N = 200 =156.80 T_m / m^2 / cm where

T_m = energy

m^2 = area

cm = penetration

2) PPR (P), (for percussion piles) = $\frac{Whn}{AP}$

W = Weight of chisel in MT

h = Fall of chisel in 'm'

n = Number or blows of hammer

A = Area of pile in 'm^2 '

p = Penetration in 'cm'

3) PPR (R), (for rotary piles) = $\frac{2\pi nTt}{Ap}$

Where

N = Revolution per minute

T = Torque in 'Tm' for corresponding 'n'

t = time in minutes

A = Area of piles in 'm^2 '

P = Penetration in 'cm'

METHOD – 1: PILE LOAD CAPACITY BY DYNAMAIC TEST USING WAVE EQUATION

The method is based on solving wave equation by using idealized model using strata wise soil parameters to arrive at pile 'set' for given pile load. The force and velocity response of pile to an impact force applied axially by drop of hammer causing large strain at top of pile (of the order of magnitude of ultimate capacity of pile) are measured to arrive at the ultimate capacity of pile.

1. THEORETICAL BACKGROUND

For piles considering resistance from surrounding soil, the internal force and displacements produced on segment of prismatic bar subjected to impact at one end, the wave equation can be derived as

$$\frac{\partial^2 D}{\partial t^2} = \left(\frac{E}{P}\right)\left(\frac{\partial^2 D}{\partial x^2}\right) \pm R$$

Where,

D = longitudinal displacement of a point of the bar from its original position

E = modulus of elasticity of bar p = density of bar material + t = time

x = direction of longitudinal axis & R = soil-resistance terms

The above equation may be solved for appropriate boundary conditions and the relationship among displacement, time, position in the pile and stress are determined usually by numerical methods. Solution requires idealization of model considering load deformation diagram for each segment, the 'quake' i.e. maximum deformation that can occur elastically. 'Ru' the ultimate soil resistance and the damping factor. The empirical value of 'quake' damping factor and percent side adhesion as reported by Forehand and Reese are reproduced for reference in **Table 1**.

Table 1 Empirical Values of Q, J, and Percent Side Adhesion

Soil	Q (in.)	J(P) (Sec/ft)	Side Adhesion (% of R_u)
Coarse sand	0.10	0.15	35
Sand gravel mixed	0.10	0.15	75-100
Fine Sand	0.15	0.15	100
Sand and clay or loam, at least 50 percent of pile in sand	0.20	0.20	25
Silt and fine sand underlain by hard strata	0.20	0.20	40
Sand and gravel underlain by hard strata	0.15	0.15	25

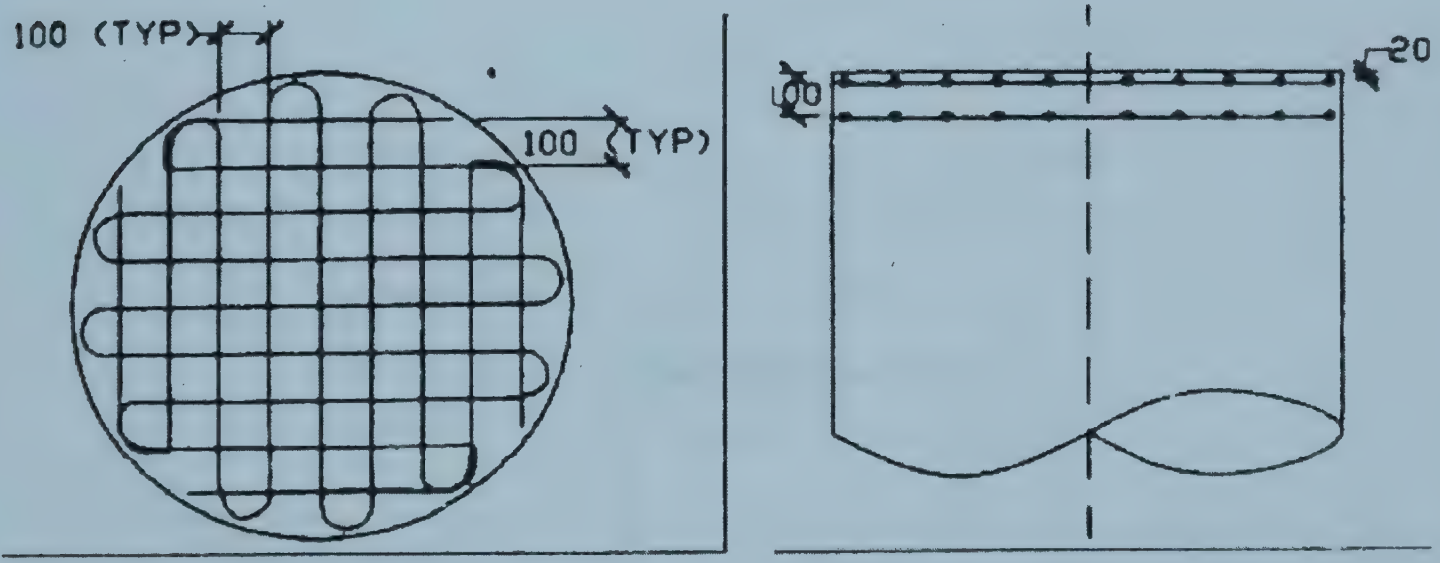
2. PILE AND TEST PREPARATION

1) The testing should be conducted by fixing instrumentation that should include strain sensors and accelerometers to the sides of the pile at a depth of 1.5 pile diameters from top of pile and then connecting them to the measuring equipment.

2) For this it is desirable that the pile is extended to suitable length after chipping top loose concrete. This can be done either using formwork or permanent casing. Alternatively, if it is a liner pile, two openings/windows approximately 300 mm x 300 mm and diametrically opposite to each other shall be made into the liner at 1.5 x pile diameter from top.

3) In case pile head is extended, it shall be axial, flat and have same strength as pile concrete. The pile head may even be one grade higher so as to attain early strength. The rebars and helical reinforcement shall also be extended to avoid cracking of concrete under hammer impact.

Refer to **Fig. 1** for a sketch of reinforcement in the extended pile and the diameter of bars shall generally be the same as pile reinforcement. Further, the concrete at the sensor level shall be smooth hard and uniform.



Plan of Test Pile Elevation of Test Pile

Fig. 1 Details of Rebar cage for extended portion of pile for dynamic test.

Note:

1. All dimensions are in mm
2. Clear cover to mesh reinforcement is 20mm
3. Diameter of mesh reinforcement bars is 8 mm
4. Spacing of mesh reinforcement bars is 100 mm

4) A pile top cushion consisting of sheets of plywood with total thickness between 25 mm to 100 mm or as determined by the Test Engineer shall be placed on the top of the pile before testing.

5) Steel helmet 25 mm-50 mm thick or as determined by the Test Engineer shall be kept ready at the time of testing.

6) A hammer of suitable weight (1-2 percent of test load or 5-7 percent of the dead weight of the pile whichever is higher) shall be used for testing the pile unless specified otherwise by the Test Engineer. The fall height generally varies from 0.5 m to 3 m.

7) Wherever essential, a suitable guide shall be provided to ensure a concentric fall.

8) A suitable crane or equivalent mechanism capable of freely falling the required hammer shall be arranged on site in consultation with the Test Engineer. Refer to **Fig. 2** showing the setup arrangements.

9) A suitable power source supply shall be provided for fixing sensors and for the test equipment.

TYPICAL SKETCH SHOWING SET UP DETAILS FOR HIGH STRAIN DYNAMIC PILE TESTING

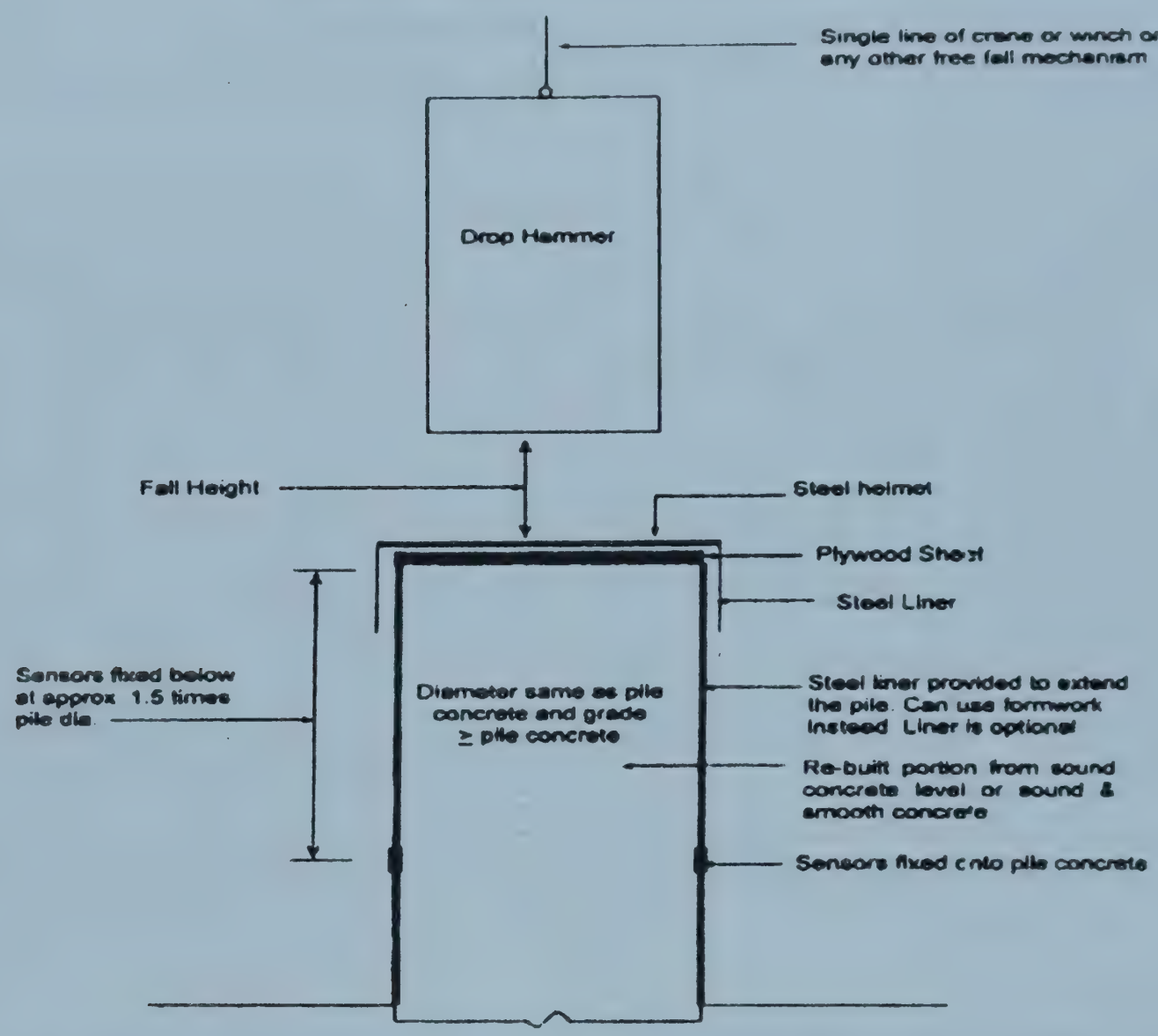

TYPICAL SKETCH SHOWING SETUP DETAILS
FOR HIGH STRAIN DYNAMIC PILE TESTING
Single line of crane or winch or any other free fall mechanism
Drop Hammer
Fall Height
Steel helmet
Plywood Sheet
Steel Liner
Sensors fixed below at approx 1.5 times pile dia
Diameter same as pile concrete and grade ≥ pile concrete
Steel liner provided to extend the pile. Can use formwork instead Liner is optional
Re-built portion from sound concrete level or sound & smooth concrete
Sensors fixed onto pile concrete

Fig. 2 Typical sketch showing setup details for high strain dynamic pile testing

3. PILE MONITORING

1) The testing may be conducted atleast 15 days after the pile is installed and the concrete pile as well as extended portion if any has achieved the required strength.

2) Dynamic pile testing (based on wave equation) should be conducted by attaching strain transducers and accelerometers to the sides of the pile approximately 1.5 times pile diameter below the pile top. A pair of transducers needs to be fixed on to opposite sides of the pile, so as to detect bending in the pile if any, during testing.

3) These transducers should be then connected through the cable to measuring equipment to record strain and acceleration measurements and display them on an oscilloscope or screen.

4) The testing should be conducted by impacting the pile with blows of the hammer, generally starting with a smaller drop of 0.5 m. For each hammer blow, the strain transducers should measure strains whereas accelerations are measured by accelerometers connected on either sides of the pile. These signals are then converted to digital form by the equipment and then converted to force and velocity respectively by integration.

5) For each hammer blow, the test system should display immediate field results in the form of the mobilized capacity of the pile, pile top compression, integrity, stresses etc. The force and velocity curve shall be generally as defined in ASTM D4945.

6) Testing should be continued by increasing height fall of the hammer by approximately 0.5 m increment till the time either the pile set or the pile capacity reaches the required limiting values. A typical force velocity response is also described in **Fig. 3**.

7) The pile capacity shall be generally considered to be fully mobilized if the energy levels due to hammer impact are sufficient so as to cause a measurable net displacement of atleast 3-4 mm per blow for a minimum three successive impacts. If the pile set is less than 3-4 mm per blow and the pile achieves required capacity then it implies that not all the static pile resistance has been mobilized and that the pile still has some capacity that could not be measured or was not required to be measured at the time of testing.

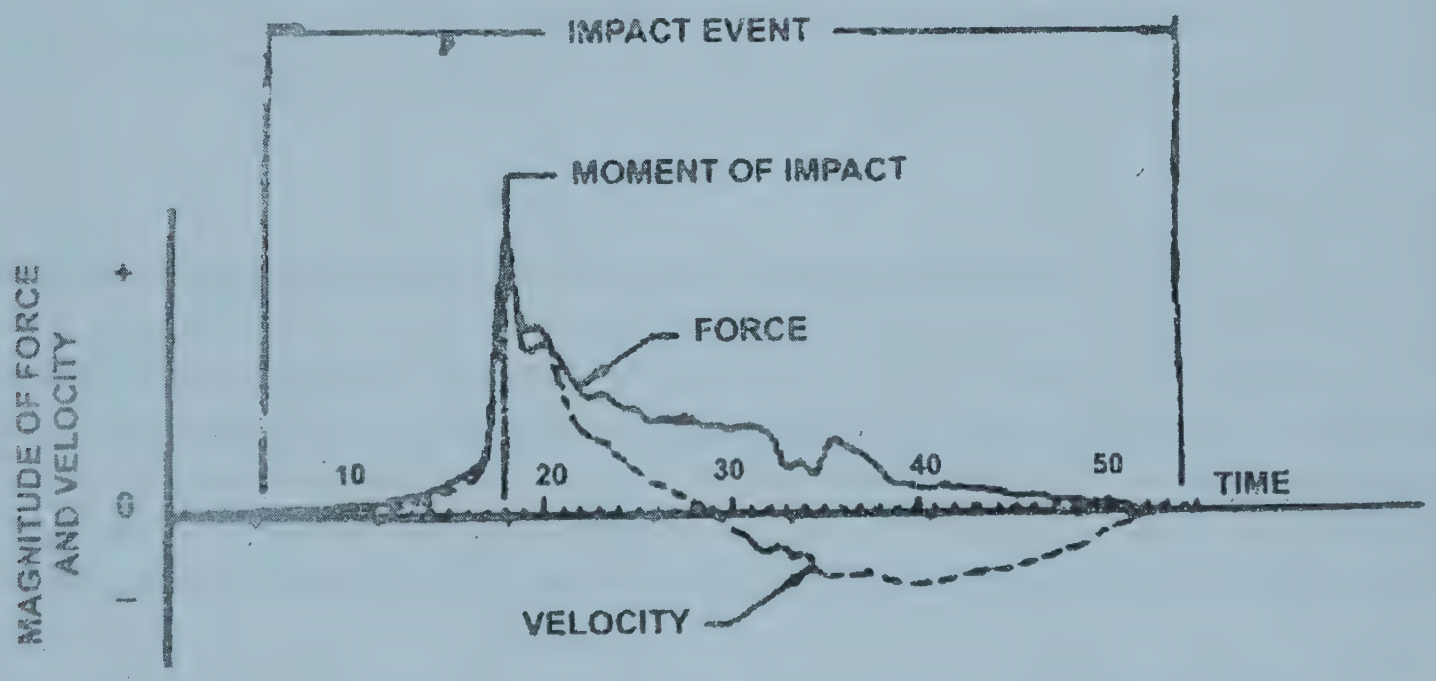


Fig. 3 Typical force velocity trace generated by measuring equipment.

8) Analysis and interpretation: Using strain and acceleration data in a suitable model, based on local parameters of pile and soil strata, the equivalent static load bearing capacity shall be calculated. The final report should specify the parameters of soil and pile strata considered and the safe capacity arrived. A Typical blow is then selected for Signal Matching Analysis.

4. TEST LIMITATIONS

1) Evaluation of static soil resistance and it's distribution can be based on a variety of analytical methods and is the subject of individual engineering judgment. The input into the analytical methods may or may not result in the dynamic evaluation matching static load test data. It is necessary to calibrate the result of the dynamic analysis with those of a static pile load test carried out according to IS:2911.

2) Based on above, it can be said that it is difficult to predict rock socket friction and actual and bearing for rock socketed piles that do not show substantial net displacement under the impacts.

3) Unlike static testing, evaluation of dynamic pile test results requires an experienced engineer trained in interpretations of the results.

METHOD – 2: PILE DYNAMIC TESTS METHOD BASED ON HILEY'S FORMULAE (BY LASER/INFRARED OPERATED EQUIPMENT)

1. INTRODUCTION

Since the early days of driven piles, the termination criteria based on "Sets observed", are followed. Various formulae are available. The IS:2911 Part – 1 covering driven piles provides one such formula. The principle followed is recording the penetration per blow of the hammer, and on that basis having obtained the desired set i.e. average penetration of standard number of blows of hammer, the ultimate capacity for the pile is worked out and then with suitable factor of safety the safe capacity is arrived at. The bored cast-in-situ piles after attaining strength (i.e. strata on which terminated) by dynamic impact energy. The load carrying capacity of bored cast-in-situ pile subjected to impact energy can be then estimated on measuring consequent displacement by sophisticated optoelectronic instruments on resorting to IS:2911 procedure. The procedure will help in ascertaining the quality of workmanship on a large number of piles without much of time wasting any avoiding delays in a construction activity with relatively less cost.

2. METHODOLOGY

The methodology of test is based on a large falling weight giving the dynamic impact to the elastic body. It equates the energy of hammer blow to work done in overcoming the resistance of the founding strata to the penetration of the ordinary cast-in-situ piles as well as grouted micro piles. Allowance is made for losses of energy due to the elastic compression of the pile, and subsoil as well as losses caused by the impact of the pile. The Modified Hiley's formulae given in the code IS:2911 Part-1, Section 1 are used in estimating the ultimate driving resistance in tones. Applying the factor of safety as outlined in the code, the safe load on pile can be worked out.

The instantaneous displacements including rebounds of the pile are precisely recorded in automatic data acquisition system. This is done for several cycles and then using formulae as accepted in IS:2911 the safe loading capacity is calculated. The opto-electronic instrument is used for position sensitive measurement by non-contact continuous measurement, using instrument placed away from the vibrations due to impact load. The system is based on combined light emitting diode transmitters and a position sensitive detector. The transmitter and receiver are installed so that the infrared light beam forms a reference line from transmitter, receiver to the prism group reflectors. The reflected light is received and recorded 100 times per second. Using the energy transmitted to the pile and accounting for temporary compressions of pile, ground and dolly occurring during the impact loading, the ultimate driving resistance is calculated. Applying lthe factor of safety the safe load for the pile is calculated.

The modified Hiley formula is:

$$R = \frac{Wh\eta}{S+C/2}$$

Where,

R = ultimate driving resistance in tones. The safe load shall be worked out by dividing it with a factor of safety of 2.5.

W = mass of the ram in tonnes

h = height of the free fall of the ram or hammer in cm taken at its full value for trigger-operated drop hammers, 80 percent of the fall of normally proportioned which operated drop hammers, and 90 percent of the stroke for single-acting hammers. When using the McKiernan Terry type of double acting hammers, 90 percent of the maker's rated energy in tonne-centimetre per blow should be substituted for the product (Wh) in the formula. The hammer should be operated at its maximum speed while the set is being taken.

η = efficiency of the blow, representing the ratio of energy after impact to the striking energy of ram

S = final set or penetration per blow in cm

C = sum of the temporary elastic compressions in cm of the pile, dolly, packings and ground

P = Mass of pile in tones

Where W is greater than Pe and the pile is driven into penetrable ground,

$$\eta = \frac{W+Pe^2}{W+P}$$

Where W is less than Pe and the pile is driven into penetrable ground

$$\eta = \frac{W+Pe^2}{W+P} - \left(\frac{W+Pe}{W+P}\right)^2$$

The following are the values of n in relation to e and to the ration of P/W

Ratio of P/W	e=0.5	e = 0.4	e = 0.32	e=.25	e=0
½	0.75	0.72	0.70	0.69	0.67
1	0.63	0.58	0.55	0.53	0.50
1 ½	0.55	0.50	0.47	0.44	0.40
2	0.50	0.44	0.40	0.37	0.33
2 ½	0.45	0.40	0.36	0.33	0.28
3	0.42	0.36	0.33	0.30	0.25
3 ½	0.39	0.33	0.30	0.27	0.22
4	0.36	0.31	0.28	0.25	0.20
5	0.31	0.27	0.24	0.21	0.16
6	0.27	0.24	0.21	0.19	0.14
7	0.24	0.21	0.19	0.17	0.12
8	0.22	0.20	0.17	0.15	0.11

P is the weight of the pile, anvil, helmet and follower (if any) in tones.

Where the pile finds refusal in rock, 0.5 P should be substituted for P in the above expression for η.

e is the coefficient of restitution of the materials under impact as tabulated below

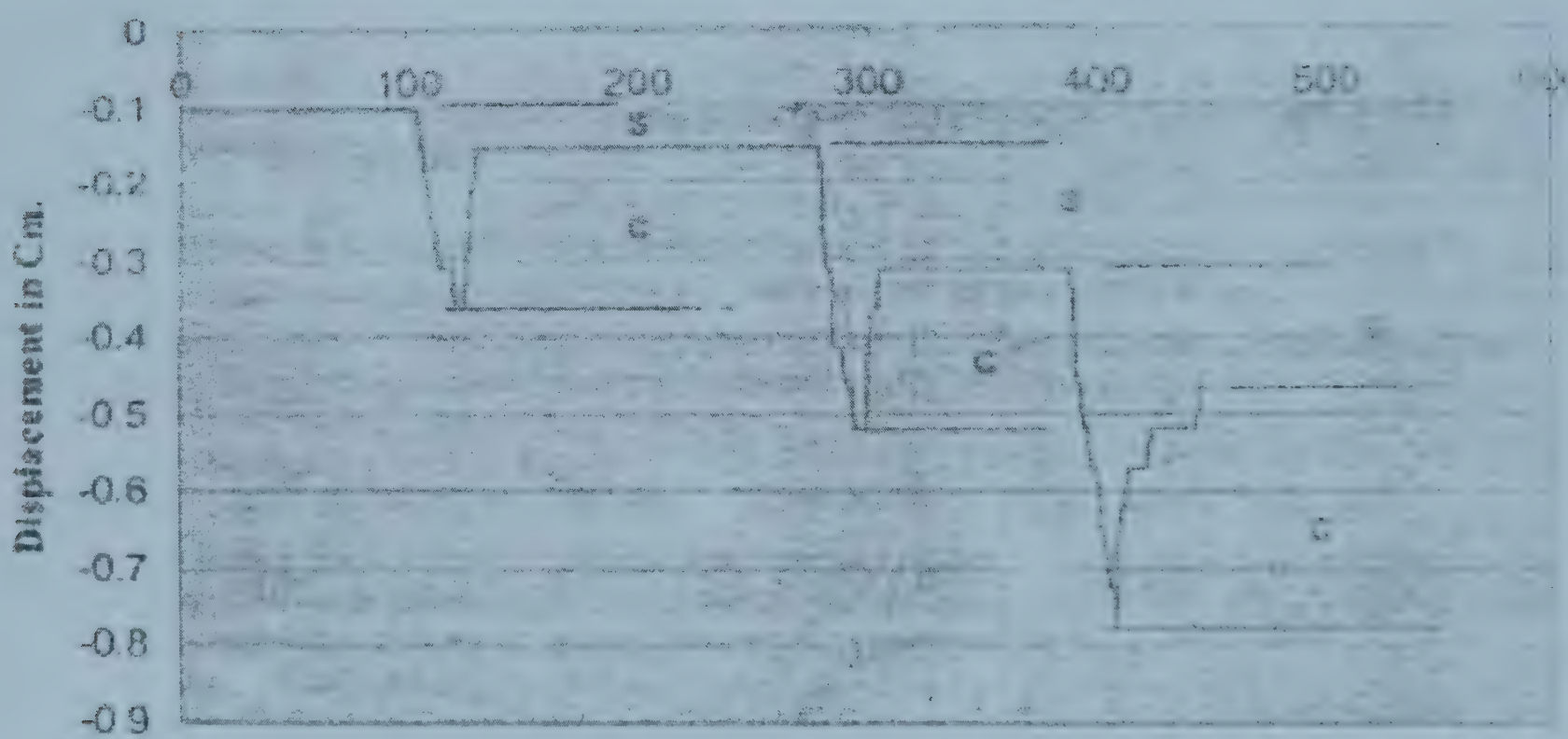
a) For steel ram of double-acting hammer striking on steel anvil and driving reinforced pile, e = 0.5

b) For cast-iron ram of single-acting or drop hammer striking on head of reinforced concrete pile, e = 0.4.

c) Single-acting or drop hammer striking a well-conditioned driving cap and helmet with hard wood dolly in driving reinforced concrete piles or directly on head of timber pile, e = 0.25.

Numbers of models of Laser/infrared operated instruments measuring accurately the deformation are available these days. The required sensitivity of the equipment shall be such as to read the angular deformation to the accuracy of 10-3 radians and the instrument should be capable of recording about 100 readings per second. From the angular deformation, on knowing the distance of the reflector from the instrument,

Typical Displacement Record



Vertical movement of the shaft under the given impact energy, (both elastic and permanent) can be measured accurately. These measurements of the displacement can then be substituted in modified Hiley's formulae stated in IS:2911. The ultimate load carrying capacity of the pile can be worked out, resorting to the modified Hiley's formulae outlined in the code and from that the safe load carrying capacity of pile can be estimated.

PART – 2
(Clause 709.2.4)

STANDARD TEST METHOD FOR LOW STRAIN PILE INTEGRITY TESTING

1 SIGNIFICANCE

Pile Integrity Testing (PIT) is a Non-Destructive integrity test method for foundation piles. The method evaluates continuity of the pile shaft and provides information on any potential defects due to honeycombs, necking, cross-section reduction, potential bulbs, sudden changes in soil stratum, concrete quality in terms of wave speed etc. It is known as "Low Strain" Method since it requires the impact of only a small hand-held hammer and the resultant strains are of extremely low magnitude. The test procedure is standardized as per ASTM D 5882 and also forms part of various specifications and code provisions worldwide as indicated in **Table 3**. The number of tests shall be decided by the Engineer to the project.

Table 3 Major Standards or Codes for Integrity Testing

S. No.	Method	Country	Reference	Title
1)	LST, CSL	Australia	Australian Standard as 21 59-1995	Pile Designing and installation
2)	LST, CSL	China	JGJ 94-94	Inspection and Acceptance of pile Foundation Engineering. 9.1 Quality Inspection of Pile installation.
3)	LST, CSL	China	JGJ 93-95	Specification for Low Strain Dynamic Testing of Piles.
4)	LST	France	Norms Francaiso	Soil investigation and testing ascnliation of buried work method by reflection impedance.
5)	LST, CSL	Germany	DGGT	Empfohlung integritataprufungen
6)	LST, CSL	UK	Institution of Civil Engineers (ICE)	Specification of Piling
7)	LST	USA	ASTM D 5882	Standard specification for Low Strain Imaginary Testing of Piles.

2 TYPICAL METHODS

Various methods used to evaluate the integrity of the pile are briefly described below. The evaluation of PIT records can be described as follows:

- Pulse-Echo Method (or Sonic Echo – a time domain analysis)
- Force Velocity Approach
- Transient Response Procedure (Frequency Domain Analysis)
- Cross Hole Sonic Logging

The Force Velocity approach is sometimes used to evaluate defects near the pile top that may be difficult to evaluate in Pulse Echo Method. The Pulse Echo is the most commonly used method and is described below:

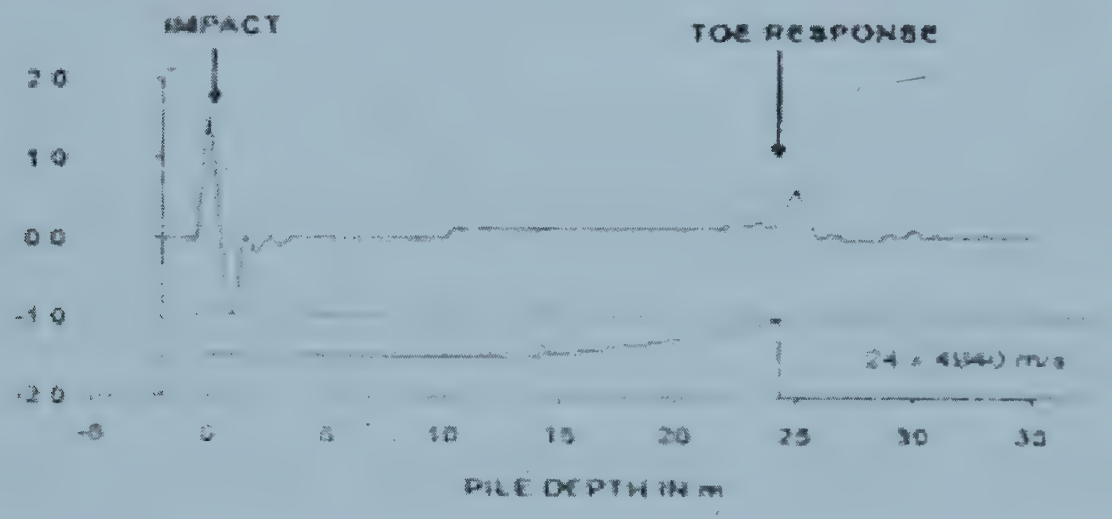
3 TEST EQUIPMENT

The test should be performed using digital data acquisition equipment like a Pile Integrity Testes or any equivalent that meets ASTM D 5882 requirements. The equipment must also include a sensitive accelerometer, instrumented or non-instrumented hammer etc. The data must be displayed in the field for evaluations of preliminary data quality and interpretation and a field printout should be possible

4 TEST PROCEDURE

The testing shall be conducted at least 7 days after pile concreting by an experienced engineer/ technician. The concrete at the pile top surface must be relatively smooth with sufficient space for attachment of the motion sensing device and hammer impact area. The testing involves attachment of an accelerometer onto the pile top (not near its edge) with the help of bonding material like candle wax, Vaseline etc. After attachment, the pile is impacted with a hand-held device (a hand-held hammer).

The test involves collection of several blows during the stage of testing. All such similar blows are averaged before display. For larger diameter piles of 600 mm and above testing may be conducted on at least 3 locations, whereas minimum one location is enough for smaller diameter piles. The typical data sets for good or damaged pile shall generally be as per Fig. 1 and is also defined in ASTM D 5882



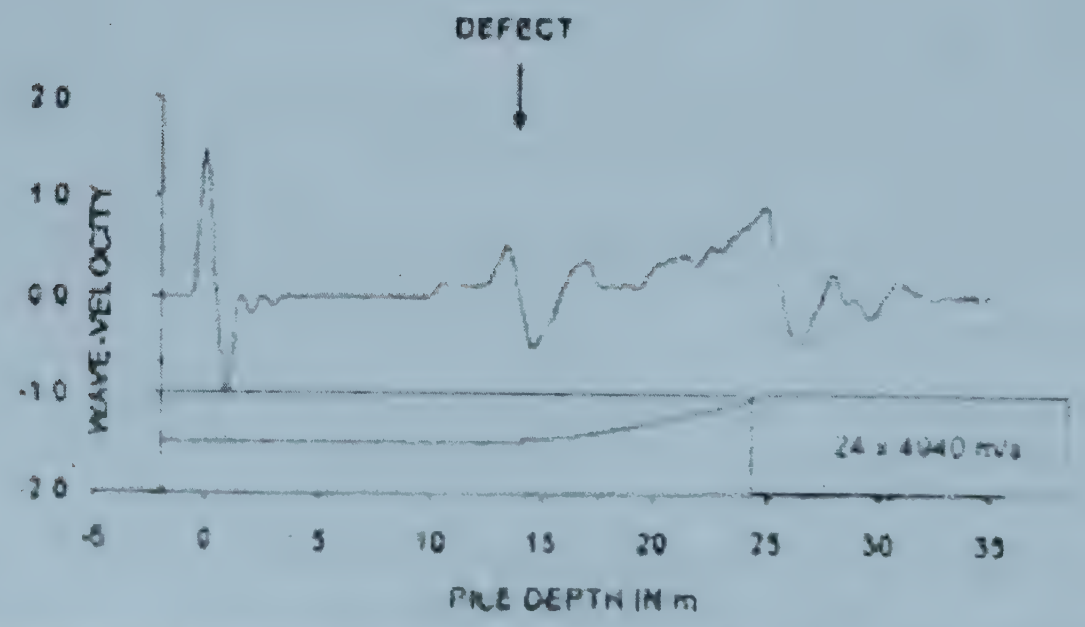


Fig. 1: PIT Velocity versus Depth plot for Normal Pile (Top) and Defective Pile (Bottom)

5 REPORT SUBMISSION

The final report shall include the following:

1) Project Identification & Location
2) Test Pile Identification including Length, Nominal Cross-Sectional area and Concrete Mix as per installation record.
3) Type of Pile and description of special installation procedure used, if any
4) Description of all the components of the apparatus for obtaining integrity measurements and recording/displaying data.
5) Graphical representation of Velocity measurements in time domain,
6) Comments on the quality of the Pile Concrete
7) Comments on any potential defects/damage and its location
8) Comments on Integrity of Pile based on above.

6 LIMITATIONS

Certain limitations are inherent in the low strain test method and hence it should be treated only as an indicator of quality of work and not as a conclusive test. The limitations mentioned below must be understood and taken into consideration in making the final integrity evaluation.

1) Integrity evaluation of a pile section below a crack that crosses the entire pile cross-sectional area or a manufactured mechanical joint is normally not possible since the impact wave likely will reflect completely at the discontinuity.
2) Piles with highly variable cross-sections or multiple discontinuous may be difficult to evaluate.
3) The method is intended to detect major anomalies and minor defects may not be detected by this method.
4) The test is not applicable to jointed pre-cast piles or hollow steel pipes or H-sections.
5) The method cannot be used to derive the pile capacity.

Appendix -7
(Clause 709.2.4)

BI-DIRECTIONAL LOAD TESTING OF PILES

1. TERMINOLOGY

Common technical terms used in this document are as below:

1.1 Bi-directional pile load test

A deep foundation pressurised by an embedded jack assembly to perform axial compressive load test, so that the foundation section above the jack assembly moves upwards and the foundation below the jack assembly moves downwards, each section providing reaction to the other. As such applied test load is twice the load in the jack assembly.

1.2 Cast-in-situ pile

A deep foundation made of concrete or cement grout and constructed in its final location like drilled shafts, bored piles, caissons, auger cast piles.

1.3 Deep foundation

A relatively slender structural element that transmits some or all of the load, it supports, to soil or rock well below the ground surface.

1.4 Jack assembly

One or more bi-directional jacks arranged together with steel bearing plates, stiffeners or equivalent to distribute jack load and to act in parallel symmetrically about a central axis. The jacks will be embedded within a deep foundation to apply a bi-directional compressive load aligned with the central axis of the deep foundation.

1.5 Bi-directional jack

A specialised hydraulic jack that has linear load-pressure calibration over its expansion range and remains accurate under small eccentric reaction conditions.

1.6 Pile reinforcement, reinforcing steel

For the purpose of this document, this may consist of any steel member such as rebar, channel, box beam, wide flange beam.

1.7 Barrette

A cast-in-situ constructed pile having non-circular cross-section, i.e. rectangular, square, T-shaped, H-shaped etc.

1.8 Telltale rod

An unrestrained metal rod extended through the test pile from a specific point within the pile to

be used as a reference from which to measure the change in the length of the loaded pile section or the absolute movement at that specific point, such as the top of an embedded jack.

1.9 Wire line

A steel wire mounted with a constant tension force between two supports and used as a reference line to read a scale indicating movement of test pile.

2. SCOPE

This Appendix provides the procedure for Bi-directional static vertical pile load testing for initial as well as routine tests for pile foundations.

This type of test is conducted on a single deep foundation element to measure load and axial displacements when loaded in bi-directional static axial compression using an embedded jack assembly. This method applies to all deep foundations, which function in a manner, similar to driven or cast-in-situ piles, regardless of their method of installation. This document provides minimum requirement for testing deep foundations under bi-directional axial vertical loads. The agency conducting the test shall interpret the results based on the procedures described in the document.

3. ADVANTAGES

This method has a number of advantages as compared to conventional Static Load Test described in IS:2911 Part IV, as below:

a) Automatic skin friction and end bearing separation

b) Eliminates heavy load frame used for load application to pile by hydraulic jacks.

c) Eliminates kenteledge platform when loads are from top of pile by sand loading or by concrete blocks. These have known to topple with attended hazards.

d) Eliminates anchor piles when reaction is taken from soil/rock below.

e) Eliminates constraint due to non-availability of area around pile due to traffic in urban areas and low head room situation.

f) Load testing can be done within cofferdam in the middle of the river or in such similar situations.

g) Saves time in preparation and load testing.

h) The total load application will be half of pile capacity, there by using lesser capacity jacks and other equipment.

i) Comparatively Safe as no loads are applied at or above ground level and very large MS frames are not used.

j) Can be used when there is space constraints

4. LIMITATIONS

The pile need to be preselected for testing and cannot be randomly selected after completion

5. TEST PRINCIPLES

5.1 The bi-directional axial vertical pile load test provides quantum of side shear parameter mobilised above embedded jack assembly and the pile end bearing parameter plus any side shear mobilised below it. Pile capacity mobilised will be two times the maximum load applied by jack assembly. Test results will provide information to assess distribution of side shear resistance along the pile and quantum of end bearing mobilised at the pile bottom, this will provide information on load-displacement behaviour of pile.

5.2 Initial test piles or test piles of bi-directional axial vertical pile tests shall be conducted for loads exceeding 2.50 times the design load or till pile fails. The parameters obtained during this test like load-displacement curves, strains etc. shall be used to optimise the designs of piles in the nearby region for their dimensions.

5.3 The initial test shall be considered as part of design for the purpose of approval. For routine piles the total loading shall be limited to the extent of 1.50 times the design load of pile. Initial pile load testing and routine pile load testing shall be in conformance with Clause 709.1.8.(C), and 709.2.4 of IRC-78 (Part-1)-2022.

5.4 Results of bi-directional vertical axial tests can be analysed to estimate the load versus movement behaviour and the ultimate capacity that could be measured during axial static compression test.

5.5 Location of jack assembly at theoretical balance point within the pile is a crucial factor to completely mobilise the axial compressive capacity, so that approximate capacity above the assembly equals the capacity below it. An improperly chosen balance point will result in excessive movement above or below jack assembly, limiting the applied load and reducing the usefulness of test results. This factor can be addressed by studying the geotechnical investigation report, so that the location of jack assembly can be at a suitable level in the piles, chosen as to ensure that all calculated upward thrust and bottom thrust due to loading will be almost equal. It shall be nearly equal with proper engineering judgement.

5.6 Estimation and establishment of balance point requires suitable site characterisation, and proper application of engineering principles and judgement. For very high capacity piles staged testing, using multiple jack assemblies at different heights will provide greater assurance of full capacity mobilisation along the complete length of pile. In case the 'balance point' demands an additional downward kenteledge load or rock anchors to ensure full mobilisation of end bearing, same shall be provided.

A schematic diagram of Bi-Directional Pile Load Testing is given in **Fig-1**

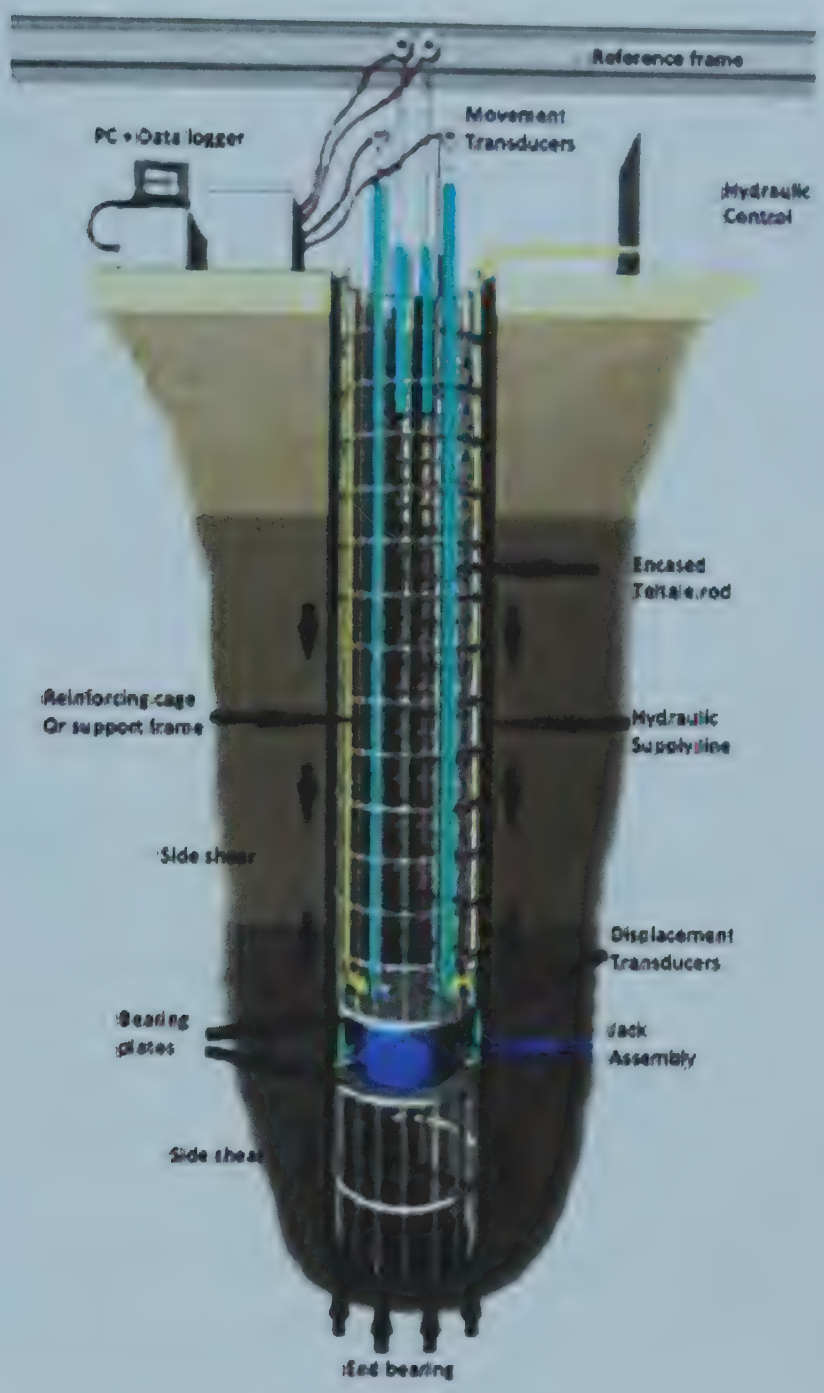


Fig. 1 A schematic diagram of Bi-Directional Pile Load Testing

6. PREPARATION OF TEST SITE

6.1 The pile under test shall be prepared such that the resultant line of force of jack assembly shall coincide with central axis of pile. **Fig. 2** to **Fig. 5** show details of embedded jack assembly and other allied details of pile and jack assembly placed within a test pile in preparation for a bi-directional axial vertical pile load testing. During the initial jack pressurisation, a fracture plane will form through the surrounding. The pile reinforcement, jack assembly and instrumentation provided shall not restrain the subsequent expansion of the assembly after the fracture occurs in the pile. The depth to the embedded jack and test instrumentation installed within the pile shall be measured to the nearest 25mm or less with respect to a common fixed point near the top of pile that will remain accessible after completion of pile installation. The test foundation preparation shall be checked and verified so that the constructed test pile will satisfy the intent of the test programme.

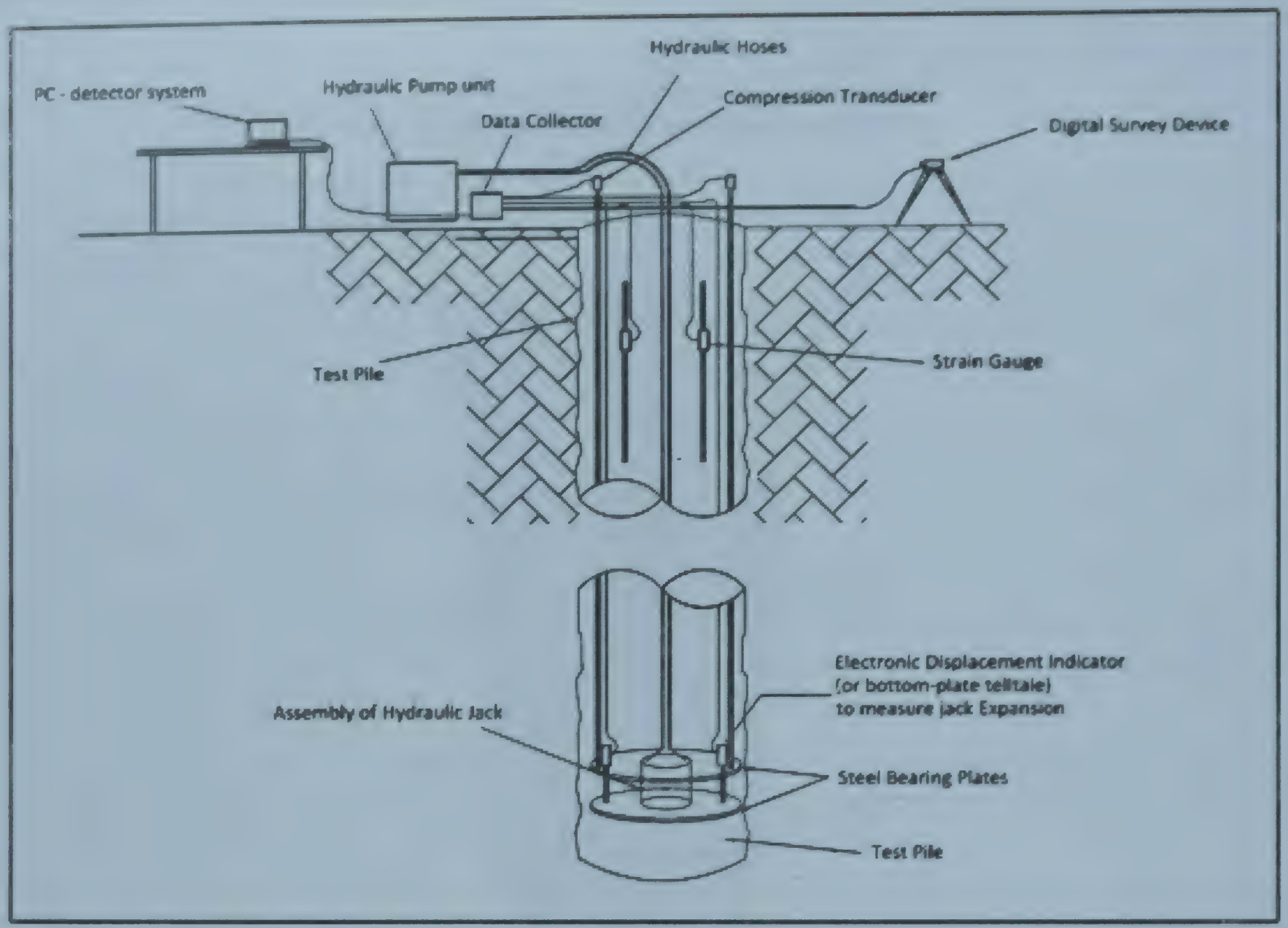


Fig. 2 Typical instrumentation above ground level

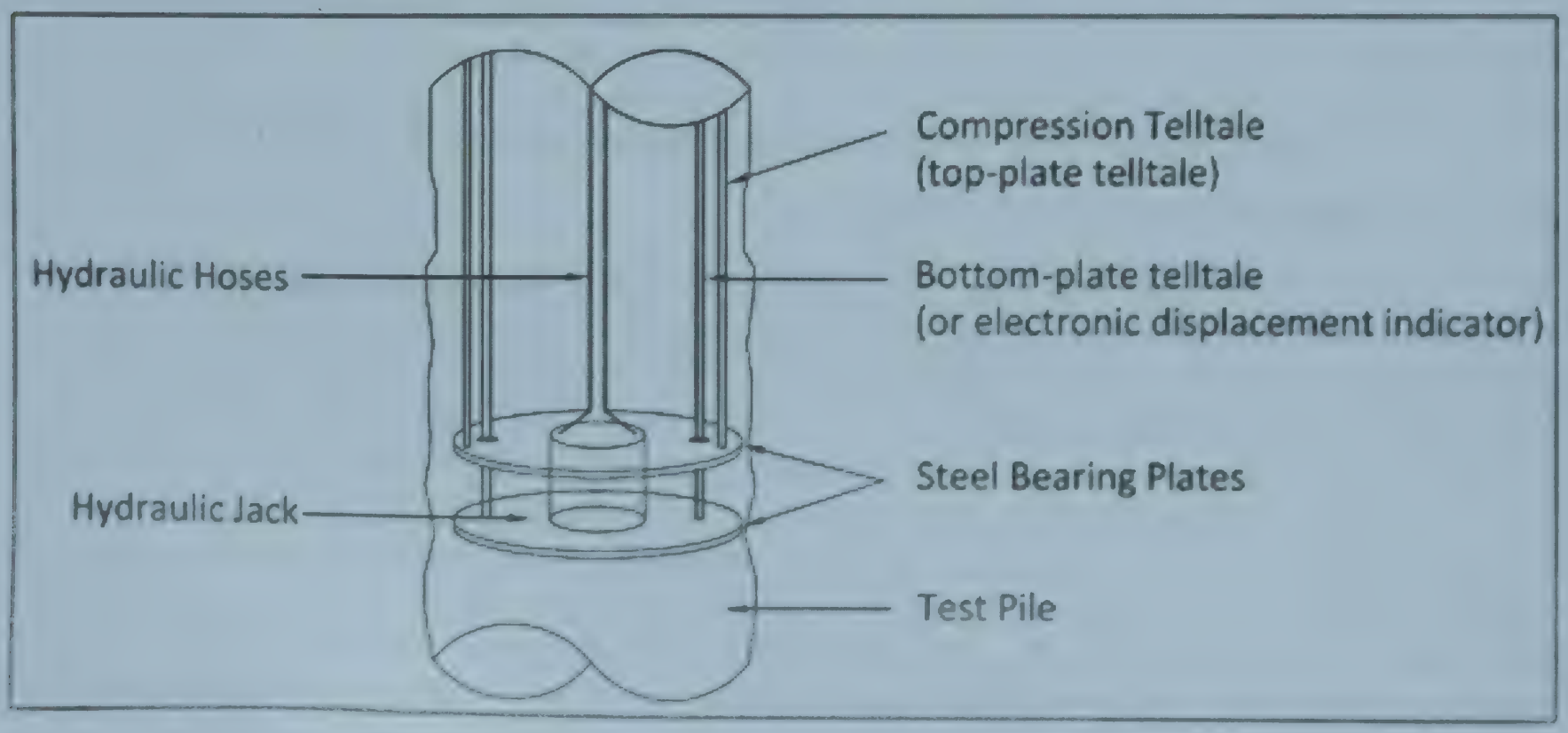


Fig. 3 Typical embedded jack movements

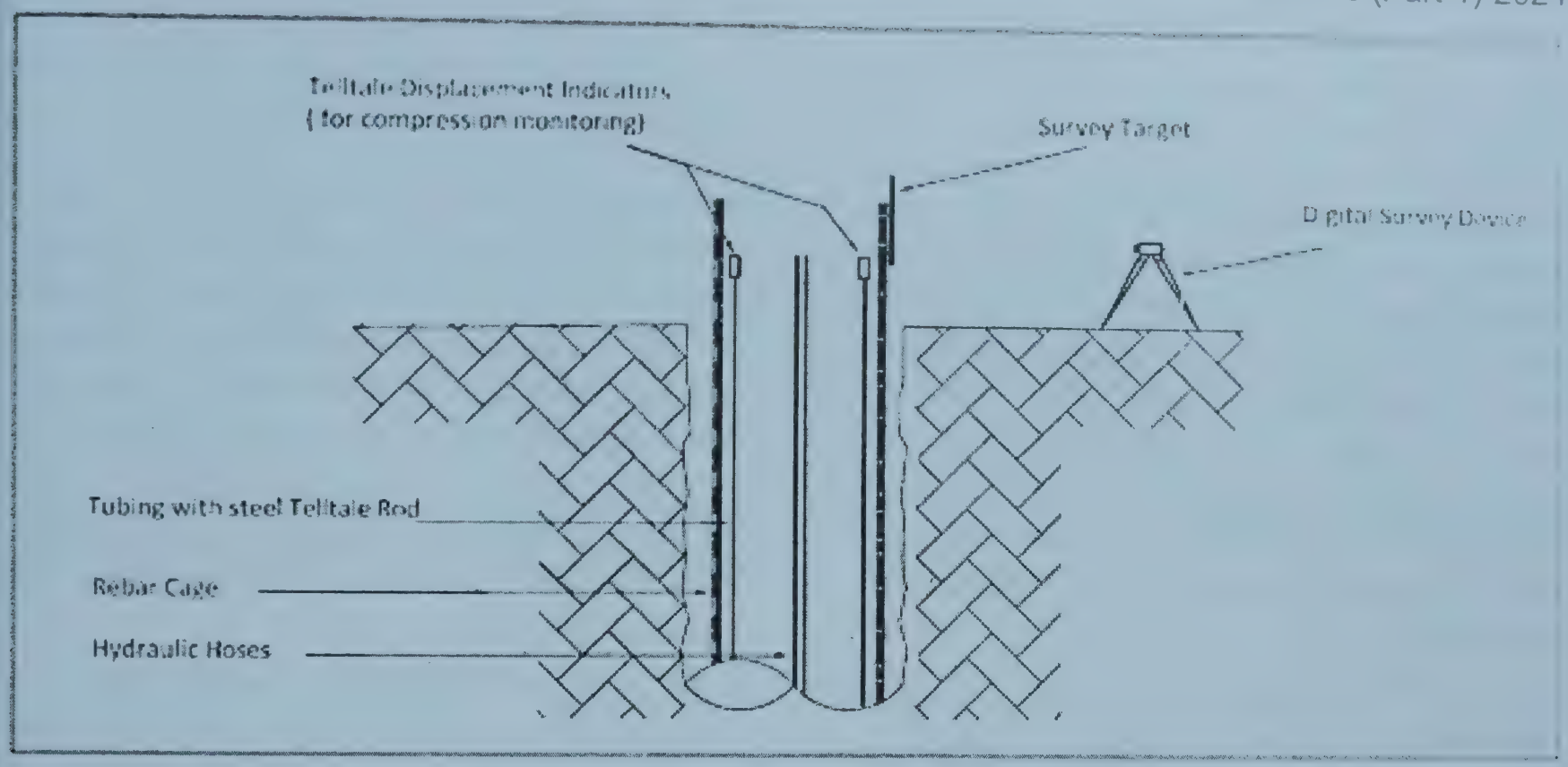


Fig. 4 Details of instrumentation for measuring pile movements

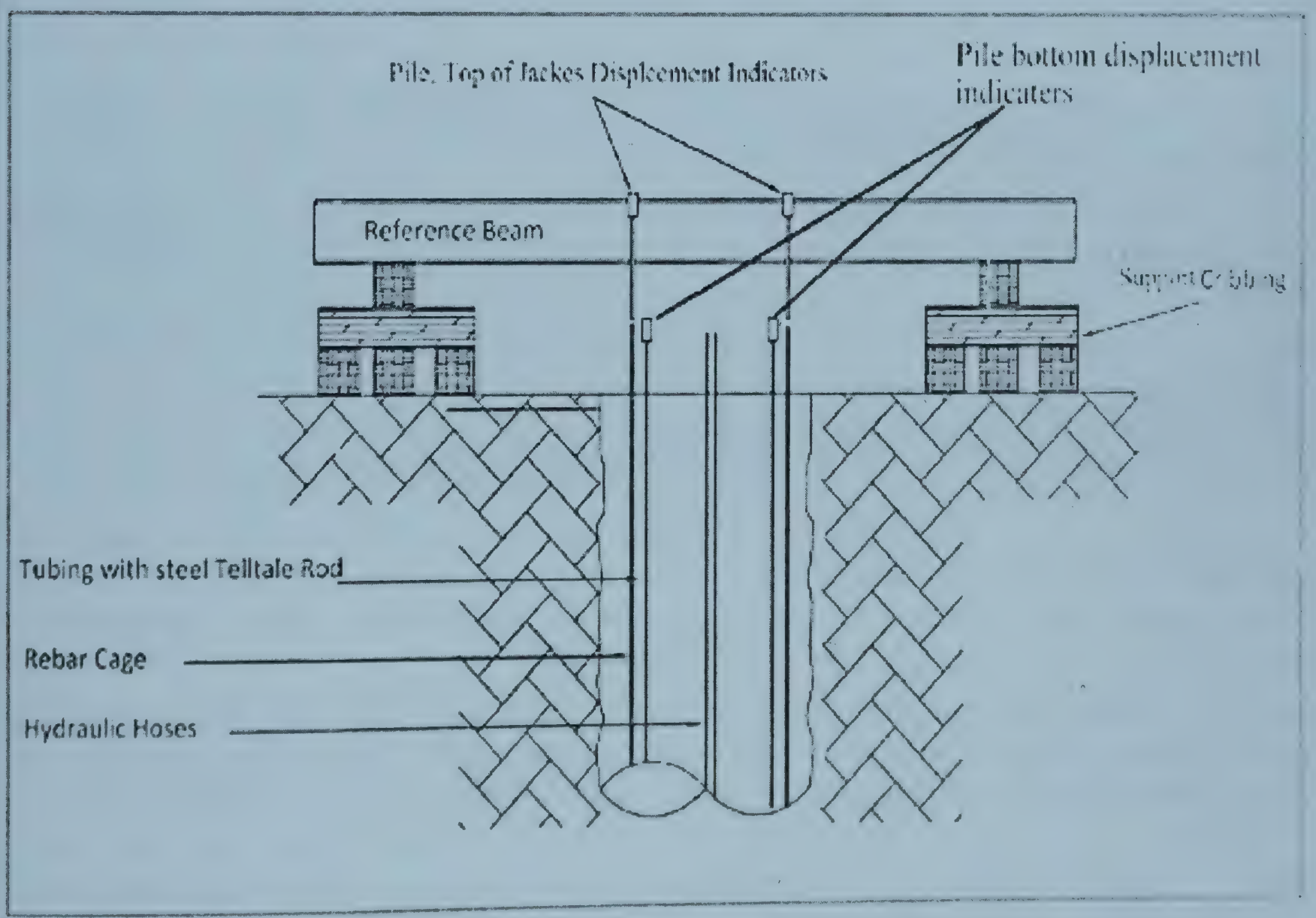


Fig. 5 Typical measurement of axial pile movement – above jack assembly

6.2 The jack assembly shall be placed at a predetermined location within the pile prior to placing the pile concrete for all types of cast-in-situ piles or drilled shafts or bored piles. Steel reinforcement cage or a similar support frame with centralised devices shall be placed to maintain the location and orientation of the assembly during concrete placement. Care shall be taken to provide for detailing of reinforcement required in the vicinity of jack assembly. Sound concrete shall be placed around the assembly by using a fluid concrete mixture, concrete shall be placed at slow and study rate, and providing adequate clearance around it to avoid restricting concrete flow and trapping any sediments, drill fluid or laitance. Drilling mud/ fluid sediments and cuttings not removed from the pile bottom before concreting can reduce the maximum possible test load if the allowable jack expansion is inadequate to compress these sediments and mobilise the planned end bearing. The jack assembly shall be placed at a minimum distance of 1.50 times pile diameter above the pile bottom, as needed to place sound concrete or grout below it. A jack assembly placed along the pile length shall provide access to place concrete beneath the assembly.

6.3 For piles constructed by auger cast method, the jack assembly shall be attached to the steel reinforcement cage or a similar support frame, and then placed into the fluid grout or concrete at the desired location. Centraliser devices shall be used to avoid damaging the jack or altering the size and shape of pile. A minimum clearance between the jack and sidewalls of bore pile shall be lesser of 75mm or 8 times the diameter of maximum size of aggregate used. The jack assembly and its support frame shall be placed in the same manner as that of piles.

6.4 For driven, pushed or screwed piles, the jack assembly is usually installed during the manufacture of pile. The pile is then installed as per normal procedure.

6.5 Jack assemblies shall be designed to load the pile symmetrically about its axis, typically using jacks of the same make, model and capacity that have individual pressure supply hoses.

6.6 A minimum of two hydraulic hoses shall be provided for input and return, extending from pile top to the jack assembly. To confirm the hydraulic flow to each jack and to isolate potential pressure leaks during the test, two hoses extending from the pile top are recommended for each individual jack. As an alternate, jacks directly connected together within an assembly shall be connected in series to verify flow continuity and check for pressure leaks. Alternatively, jack can be connected in parallel with an embedded manifold to verify the flow and pressure to each jack. To limit potential leaks, hoses should not include unnecessary fittings or connections within the pile. Each hose shall be clearly marked at each jack, at both sides of any connection and at the pile top to identify the jack connected to it.

6.7 Routine piles or working piles can use the vent pipes or tubes for post-test grouting of fracture plane created in the pile by the expanded jack assembly. The hydraulic hoses installed for each jack is used to grout the expanded jacks. Grout used shall be of non-shrink cementitious in nature.

6.8 The ground level around the test pile shall be excavated or filled up to final design elevation. Cut off or build up the test pile as necessary to permit the placement, use and operation of test equipment and instrumentation. Any damaged or unsound material shall be removed from pile top as required to install instrumentation for testing. While temporarily dewatering the

site, a ground water level shall be maintained as near to the nominal elevation as possible for the duration of test.

7. PLANNING OF LOAD TESTING

7.1 The maximum anticipated load applied by the embedded jack assembly in each direction will be half of the specified maximum test load. The rated capacity of the jack assembly shall exceed the maximum anticipated load by 25%. The jack assembly shall be pressurised using hydraulic fluid oil or water or both. The ram of each bi-directional jack shall have a minimum travel of 150mm or it can be less than 150mm if the test demands for the same. However, it is preferred to have ram travel of a minimum of 150mm.

7.2 **Location of jack assembly for routine load test:** The jack assembly shall be below the designed length of pile and in continuation to the pile.

7.3 Bi-directional jacks shall be assembled together with mild steel bearing plates, stiffeners or equivalent, so that jack load can be distributed to outer perimeter of the pile. Bearing plates and stiffeners shall provide a uniform distribution of the axial load and allow for smooth flow of concrete below the jack assembly and into the assembly between bearing plates. Distance from pile top reference to the jack assembly top and bottom shall be measured and recorded to nearest 25mm or less. A Jack assembly consists of steel plates fixed to the required no. of jacks. Refer **Fig. 6** and **Fig. 7**.

Fig. 6 Jacks being assembled with steel bearing

Fig. 7 Assembled arrangement readyplates to be installed in pile

Note: Cutouts in steel bearing plates for passage of concrete by tremie are shown in **Fig. 6** and **Fig. 7**.

7.4 Jack assembly shall be welded or locked so that it remains closed during handling and installation in the pile. The welds or locks shall be designed, so that, they can be disengaged completely with no resistance to expansion of jack, prior to testing or to provide resistance after 1mm or less of assembly expansion. When placed as integral part of the pile reinforcement in a cast-in-situ pile, the jack assembly and its connection to the reinforcement shall be designed to safely withstand handling and placement stresses. Jack assemblies are introduced in the steel reinforcement cage at the required location. As such hinge is introduced in upper portion near jack assembly, refer **Fig. 8** and **Fig. 9**.

Fig. 8 Jack assembly attached to pile

Fig. 9 Jack assembly attached to pile

7.5 Jack assembly casing for driven pile shall include anchorage that will safely withstand handling and driving stresses. Refer **Fig. 10**.

Fig. 10 Lowering of assembly with cage reinforcement

Bi-directional jacks that opens within the soil, especially when installed in a driven pile, can include a plate around the pile perimeter that covers the opening to minimise disturbance of the surrounding soil during testing.

7.6 The pump, hoses, pipes, fittings, pressure gauges and pressure sensors used to pressurise the jack assembly shall be rated to a minimum safe pressure corresponding to the nominal capacity of the jack assembly.

7.7 The pressure measurements in the jack assembly using pressure gauges or sensors shall have a range greater than or equal to the rated pressure of the jack assembly. Pressure gauges shall have minimum graduations less than or equal to 1% of maximum anticipated pressure. When used in the test, pressure sensor output in units of pressure or calibrated load shall be displayed in real time during the test.

7.8 Each jack, pressure gauge and pressure sensor shall be plainly marked by a unique serial number and shall have calibration tests performed no more than twelve months prior to the test to atleast the maximum anticipated jack pressure.

7.9 Strain gauges are recommended at strategic locations to assess the load in the pile and the load transfer along the pile length, which facilitates the equivalent top-down load-displacement analysis.

8. MEASUREMENT OF DISPLACEMENTS AND STRAINS

8.1 Upward pile movement shall be reported as positive and downward movement as negative. Pile compression shall be reported as positive and expansion as negative. Jack assembly expansion shall be reported as positive and closure as negative.

8.2 Reference beams and wire lines, if used, shall have supports firmly embedded in the ground at a clear distance from the test pile of atleast three times the diameter of test pile. A single reference beam shall be oriented across pile top or two parallel reference beams, one on each side of test pile. Reference beams shall have adequate strength, stiffness and cross bracing to provide stable support for the test instrumentation and to minimise vibrations that may affect the measurement of pile movement. One end of each beam shall be free to move laterally as the beam length changes with temperature variation. Supports for reference beams and wire lines shall be isolated from moving water and wave action.

8.3 A trap or shelter shall be provided to prevent direct sunlight and precipitation from affecting the measuring and reference systems. Refer **Fig. 11**.

Fig. 11 On-site test set up location

8.4 The dial indicator stem shall be perpendicular to the direction of stem travel and bearing surface shall be smooth with atleast a 5mm glass plate glued to the surface. Electronic indicator movements shall be displayed in real-time during the test. Displacement indicators used for measuring pile movements shall have a minimum travel of 100mm and minimum graduations of 0.01mm or less. If larger displacements are anticipated, greater travel, stem extensions shall

be provided. Dial indicators and electronic displacement indicators shall be in good working condition and shall have a full range calibration within twelve months prior to the test.

8.5 Laser, optical or digital survey levels can be used for secondary pile top axial movement measurements and to verify reference movements.

8.6 Unstrained telltale rods, with a typical diameter of 6mm shall be used to measure the axial pile movement or the axial compression within the pile. Telltale rods shall be installed in an open sheath or casing or tubing having an inside diameter approximately two times the telltale rod diameter to ensure free rod movement during the test. A displacement indicator shall be used with its stem parallel to the pile axis to measure the relative movement between the rod and the pile top or the reference beam. A glass plate shall be clamped and glued to the telltale rod perpendicular to the rod. A single telltale can be installed on the axis or telltales can be provided in pairs at the same elevation to obtain an average measurement on the pile axis, with the telltales in each pair oriented diametrically opposite to each other and equidistant from and parallel to pile axis. For test piles exceeding 1.80m diameter, a minimum of two pairs of telltales shall be installed at each elevation to obtain an average measurement, ideally with one pair orthogonal to other. The telltale rods shall have a rounded or pointed tip that bears on a clean steel plate affixed within the pile or shall be threaded into a nut affixed within the pile or firmly fixed with any available method. Telltale rods shall be cleaned and oiled prior to installation in pile. Centralisers shall be provided for the rods at the pile top to restrain lateral movement but not axial movement. Alternatively, a axial tension displacement indicator can be attached to the telltale and other end can be fixed to a rigid reference to measure the axial movement of telltale rod. Distance from pile top reference to the termination point of each telltale shall be measured to nearest 25mm or less.

8.7 Each displacement indicator, scale, target, detector, staff and reference point used during the test shall be identified and marked with reference numbers or letters clearly visible to test personnel.

8.8 Indicators, scales or reference points attached to the test pile, reference beam or other references shall be firmly affixed to prevent slippage during test. Verification shall be done for wire line supports and reference beams, so as, to prevent movement during the test by using a surveyor's level for taking readings on a survey rod or scale with reference to permanent bench mark located outside immediate test area.

8.9 Axial movements on pile top

Displacement indicators shall be used as primary or secondary system to measure pile top axial movement with respect to one or more reference beams. The indicator stem shall be oriented parallel to the pile axis. A single displacement indicator shall be mounted on a reference beam to measure axial movement at the centre of the test pile. As an alternate, displacement indicators shall be mounted on reference beams in pairs to bear on pile top at opposing axisymmetric points equidistant from the centre of test pile. During the test, use a level or laser with scales, targets, detectors or staff to measure the movement the reference beam relative to a benchmark located outside of immediate test area.

8.10 Axial movements of Jack assembly

Telltales shall be installed to measure respective movements of the top and bottom of jack assembly. Displacement indicators used to measure these movements shall have adequate length to measure assembly plate movement and shall measure the axial movements of the jack assembly to the nearest 0.01mm. Telltales shall be referenced to the top of the pile or to a reference beam system. Monitor the axial expansion of the jack assembly using the difference between the telltale measurements at the top and bottom of the assembly.

8.11 Direct jack expansion measurement

Electronic displacement indicators shall be installed in the pile to directly measure the jack assembly expansion. These electronic displacement indicators shall measure assembly expansion to the nearest 0.01mm. A minimum of one pair of indicators shall be installed, with the indicators in each pair oriented diametrically opposite to each other and equidistance from and parallel to pile axis. For test pile exceeding 1.80m diameter, a minimum of two such pairs of indicators shall be used. In addition, at least one pair of telltales shall be used by extending from pile top and up to telltale level coming from top of jack assembly to measure the pile compression above the jack assembly.

When the jack assembly is activated, the lower and upper portions of the jack will move in the respective directions with displacements. A typical pile having length of 25000mm with displacement of 50mm upward and 50mm downward movement is shown in **Fig. 12**. Telltales-TT-1, TT-2 are for upward movement of pile and Telltales-TT-3 and TT-4 are provided for downward movement of pile below the jack assembly. The lengths given in the **Fig. 12** are notional only and for the purpose of understanding.

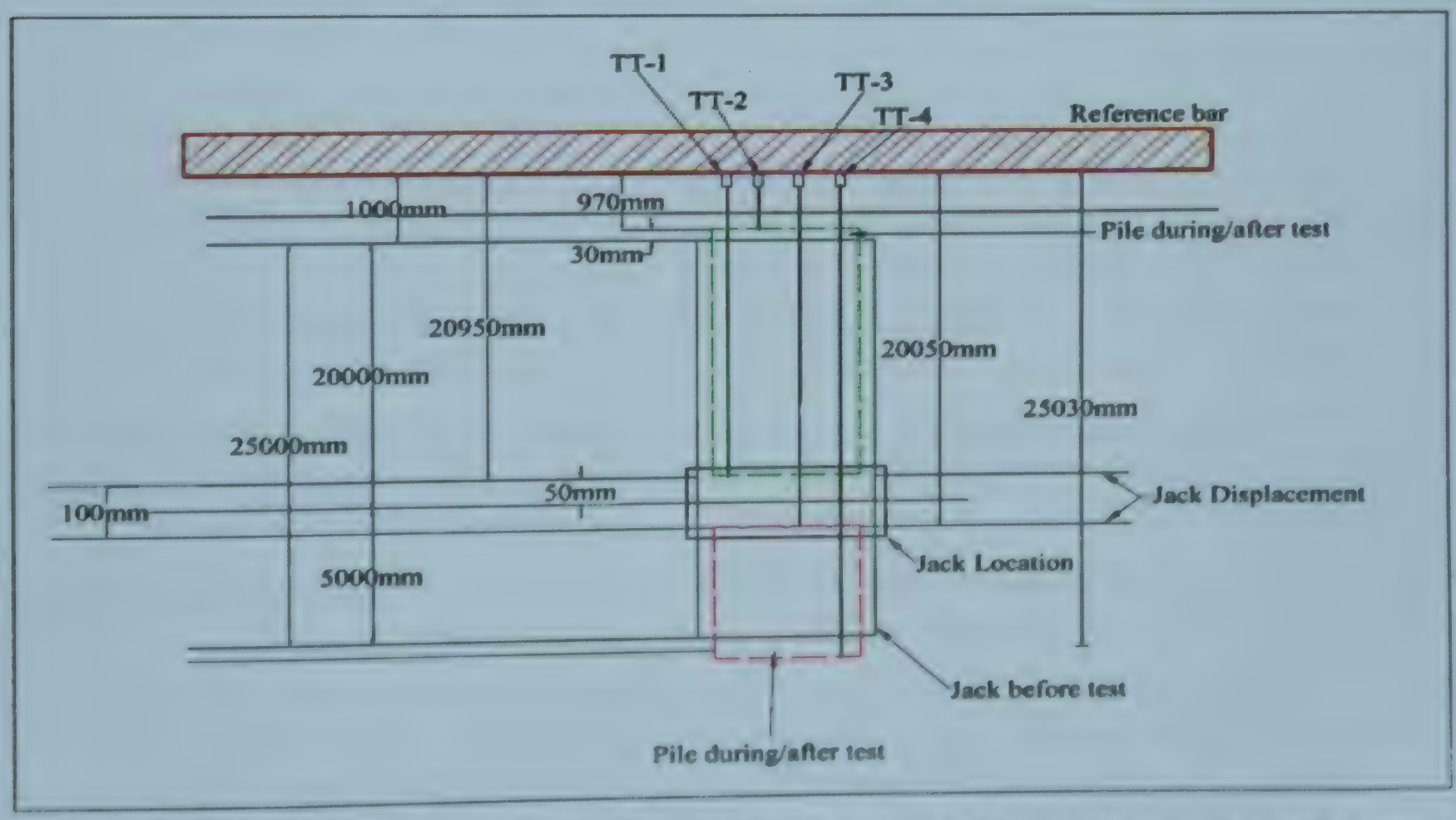


Fig. 12 A typical jack assembly movement when activated for a pile length of 25m. The measurements are notional only and for the purpose of better understanding.

8.12 Strain measurement in pile

Wherever strain measurements are envisaged, strain in the pile can be measured using strain gauges installed along the length of pile axis at locations specified to help evaluate the distribution of load transfer from the pile to the surrounding soil. Single gauges or paired gauges can be installed on pile axis with the gauges in each pair oriented symmetrically opposite to each other and equidistant from and parallel to the pile axis. Two such pairs of gauges are recommended for critical locations and for test piles with diameter exceeding 1.80m or larger, ideally with one pair orthogonal to the other. Distance from pile top reference to the gauges is measured and recorded to the nearest 10mm or less.

The strain gauges shall be individually or batch calibrated prior to installation and shall have accuracy within 1% throughout the applicable measurement range. Strain readings shall be recorded with a precision of 1 micro strain or less. Where feasible, strain measurement programme shall include a complete history of gauge readings starting before their installation in the pile.

In order to interpret strain measurement and to estimate the load in the pile, details regarding depth profile describing the variation and properties of pile constituents, including strength, cross-sectional area and modulus obtained are required. The modulus can vary with the applied stress, especially for grout or concrete. This information can be obtained from installation records, calliper measurements and separate material property tests as needed.

8.13 Pile compression measurements

Wherever pile compression measurements are envisaged, same shall be measured in test pile.

9. LOAD TESTING PROCEDURE

9.1 General

Use of embedded jack assembly to apply load to the test pile will result in application of test load which is twice the load measured in jack assembly. The test uses load intervals to better estimate the anticipated movements. Structural capacity of pile or jack assembly shall not be exceeded in the test procedure.

9.1.1 Prior to testing, concrete in the pile should generally achieve approximately uniform strength throughout the pile and atleast 85% of maximum design compressive strength.

9.1.2 The static axial capacity of piles typically changes as time elapses after pile installation, depending on the soil or rock properties, on the pore water pressure and soil structure interface while installing pile. This behaviour can be for both cast-in-situ as well as driven piles.

9.1.3 Prior to performing the test hydraulic fluid shall be circulated through each jack to verify hose connectivity, saturate the system and flush any blockages.

9.1.4 Prior to test, any safety locks placed on the jack assembly shall be removed for safe handling during placement in the pile. Welds must be broken during initial pressurisation of the jack assembly. The initial pressurisation shall proceed until a fracture plane forms across the pile and all welds or locking mechanisms are fully disengaged, at that time the pressure shall be reduced to zero for atleast 1 minute before beginning the loading on pile.

9.1.5 Unless otherwise specified each increment of test load shall be maintained constant with reasonable tolerance.

9.1.6 Jack operating personnel shall check for pressure leaks during each load interval. Continuous pumping without expansion of jack assembly or a significant difference between the pressures measured on the input and return pressure lines can indicate hydraulic leak. Leaks shall be identified, isolated and repaired before continuing the test. The effect of leak shall be assessed with reliability of test results.

9.2 Loading procedure

9.2.1. Load on jack assembly shall be applied in ten equal increments with each increment not more than 5% of maximum specified test load, since maximum specified jack assembly load will be 50% of maximum specified test load. Each load increment shall be added in a continuous operation attaining the load following the completion of movement readings for the previous load intervals. If significant movement occurs above or below the jack assembly, the loading time is extended to apply the load increment to the opposing portion of the pile. Load increments shall be added till reaching half of the maximum specified test load, until reaching the maximum expansion or load capacity of jack assembly, or until observing continuing, progressive expansion of jack assembly, but structural capacity of pile shall not be exceeded. Below template shows Load vs displacements - top and bottom movements.

Template

Load (kN)	Top plate movement (mm)	Bottom plate movement (mm)	Total (mm)	Remark

Note: Use TT-1 and TT-3 from **Fig. 12** for measuring and Plotting load vs displacement table and curves

9.3. Recording test readings

9.3.1 Time of applied pressure, jack assembly load, pile movement, jack assembly expansion, pile compression, pile strain, reference movement for each identified gauge, scale and reference point shall be recorded for each increment immediately, preferably within 1 minute of each loading.

9.3.2 It is preferable to use a data logger system to condition and store the test data in digital form. The data logger shall be connected to an electronic system that displays the numeric results in real time during the test.

9.3.3 The jack's manufacturers shall certify its accuracy.

9.3.4 Load shall be applied in each increment and decrement of 5% of maximum specified test load at 1, 2, 5, 10 and further each at 10 minutes upto the duration of total load. It should be noted here that maximum specified jack assembly load will be 50% of maximum specified test load in bi-directional testing. Instrument reading shall be recorded within 1 minute before application of next load increment or decrement. If required, data logger can be used to automate the test.

10 SAFETY DURING TESTING

i) All operations in connection with pile load testing shall be carried out, so as to, minimise or avoid or eliminate exposure of people to any type of hazard due to test.

ii) Stable and level work areas shall be provided around the test pile. All test and adjacent work area's walkways, platforms shall be cleared of scrap, debris, small tools and accumulated mud, grease, oil and other substances.

iii) Temporary devices to keep embedded jack assembly safely closed during handling and placement is provided. When placing in jack assembly as part of steel reinforcement cage, adequate connections between steel reinforcement and jack assembly shall be provided to maintain stability and integrity of overall cage during its handling and placement. Multiple lifting connections shall be used to prevent permanent distortion of reinforcement cage.

iv) Loads shall not be hoisted, swung or suspended over test personnel and shall be controlled by tag hoists. Only authorised personnel and test equipment shall be permitted within immediate test area.

11 ANALYSIS OF RESULTS

11.1 Measurements obtained in the form of deformation upwards and downwards shall be plotted as in **Fig. 13**. The ordinate above 0.00 mm displacement with respect to the load in abscissa indicate upward movement of the jack system recorded, and the ordinate below 0.00 mm with respect to the load in abscissa indicate downward movement of the jack system at respective loads. For any particular load combined movement measured above 0.00 abscissa line as well as below the line will indicate total displacement of the jack assembly. Further an equivalent top load curve derived from **Fig. 13** and plotted as in **Fig. 14**. This equivalent top-load curve shall be read for load – displacement requirements.

11.2 Initial Load test or Load Test on Test Pile

Test pile or load test on Initial pile shall be conducted for loads exceeding 2.5 times the design load and capacity measured corresponding to settlement of 10% of pile diameter or less if the settlements is less than 10% of pile dia.

11.3 **Routine Load Test** shall be conducted for test load of 1.50 times design load and settlements shall not exceed settlement obtained from initial pile test corresponding to routine test load.

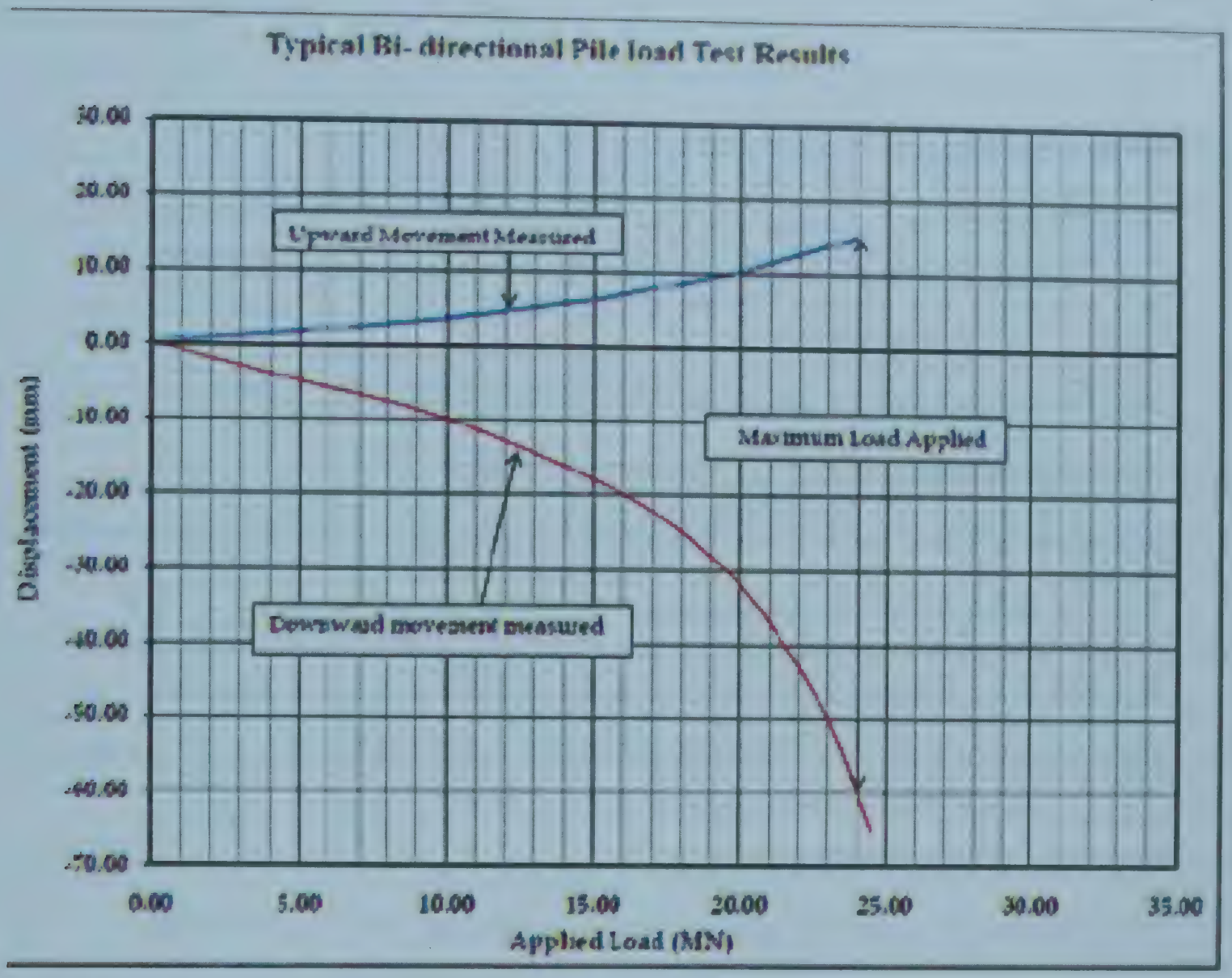


Fig. 13 Typical individual top-bottom displacement of pile with respect to load

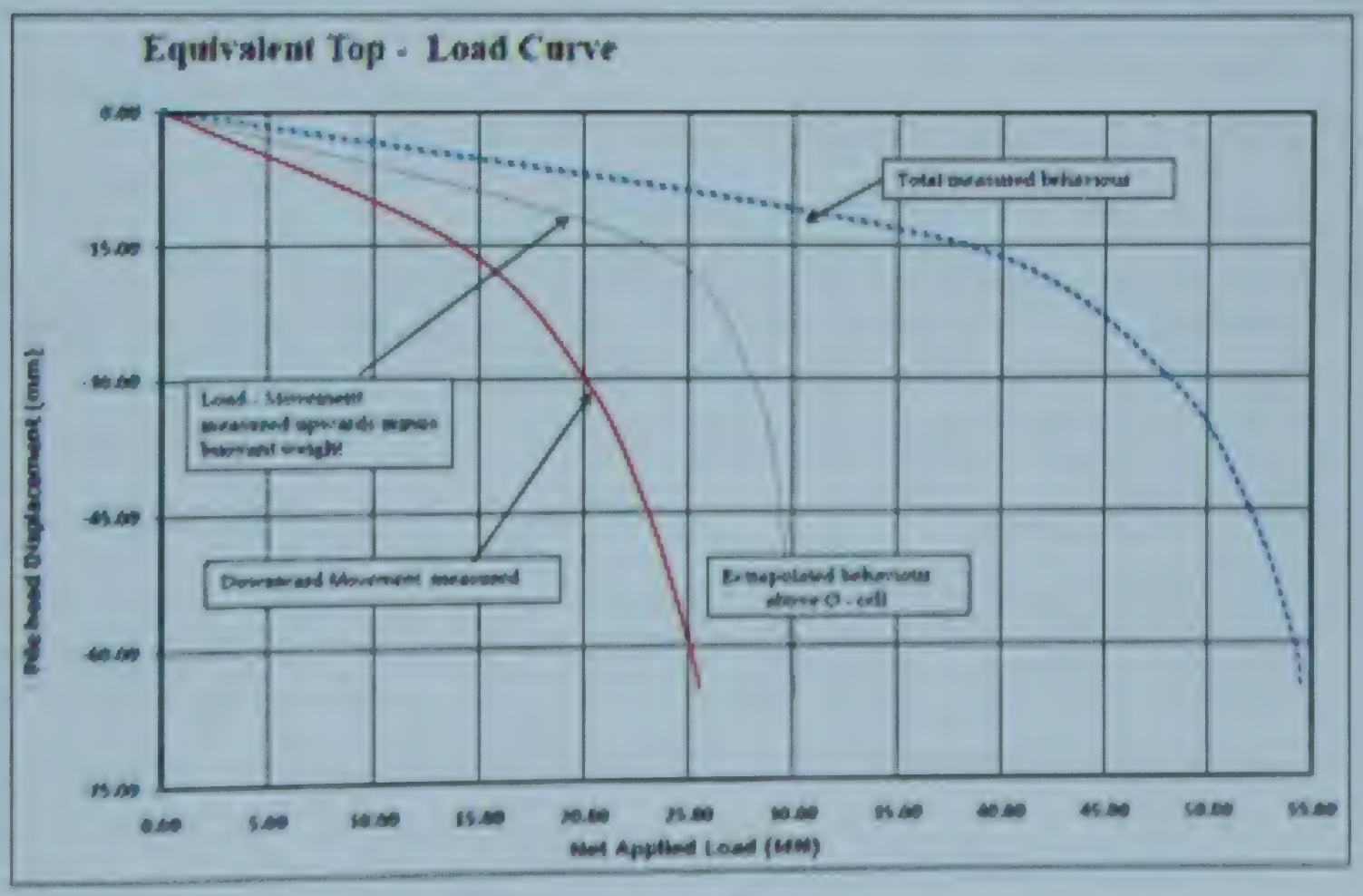

Equivalent Top - Load Curve
Total measured behaviour
Load - Movement measured upwards minus buoyant weight
Downward Movement measured
Extrapolated behaviour above O - cell
Pile head Displacement (mm)
Net Applied Load (MN)

Fig-14 Typical equivalent combined curve from load test

12 REPORT

Test information shall be recorded for

i) Project identification and location.

ii) Test site location

iii) Date and type of test

iv) Agency performing the test

v) Personnel performing the test and preparing the report

vi) Temperature and weather conditions during tests

vii) Brief description of embedded jack assembly and pressure measurements, including capacity

viii) Description of instrumentation used to measure pile movement including location of indicators, scales and other reference points with respect to pile top.

ix) Description of special instrumentation such as telltales or strain gauges including location of such instruments with reference to pile top

x) Special testing procedures used

xi) Plots of load vs plate movement above and below the jack assembly

xii) Photographs of test instrumentation and setup (optional)

xiii) Date test pile installed

xiv) Design load of test pile

xv) Type and dimensions of test pile to nearest 0.03m or less.

xvi) Test pile material including basic specifications.

xvii) Load vs displacement curves for upward and downward movement of pile shall be recorded, equivalent top load curve from earlier curves shall be plotted, further strain gauge readings wherever required shall be plotted.

Appendix-8

(Clause 710.1.4)

FILLING BEHIND ABUTMENTS, WING AND RETURN WALLS

1 FILLING MATERIALS

The type of materials to be used for filling behind abutments and other earth retaining structures, should be selected with care. A general guide to the selection of soils is given in **Table 1**.

Table 1 General Guide to the Selection of Soils on Basis of Anticipated Embankment Performance

Soil group according to IS 1498-1970 Most probable	Possible	Visual description	Max. dry density range kg/m_3	Optimum moisture content range percent	Anticipated embankment performance
GW, GP, GM, SW, HP		Granular materials	1850-2280	7-15	Good to Excellent
SB, SM, GM, GC, SM, SC		Granular materials with soil	1760-2160	9-18	Fair to Excellent
SP		Sand	1760-1850	19-25	Fair to Good
ML, MH, DL	CL, SM, SB, SC	Sandy Silts & Silts	1760-2080	10-20	Fair to Good

2 LAYING AND COMPACTION

2.1 Laying of Filter Media for Drainage

The filter material shall be well packed to a thickness of not less than 600 mm with smaller size towards the soil and bigger size towards the wall and provided over the entire surface behind abutment wings or return walls to the full height.

Filter materials need not be provided in case the abutment is of spill through type.

2.2 Density of Compaction

Densities to be aimed at in compaction shall be chosen with due regard to factors, such as, the soil type, height of embankment, drainage conditions, position of the individual layers and type of plant available for compaction.

Each compacted layer shall be tested in the field for density and accepted before the operations for next layer are begun.

3. EXTENT OF BACKFILL

The extent of backfill to be provided behind the abutment should be as illustrated in Fig. 1

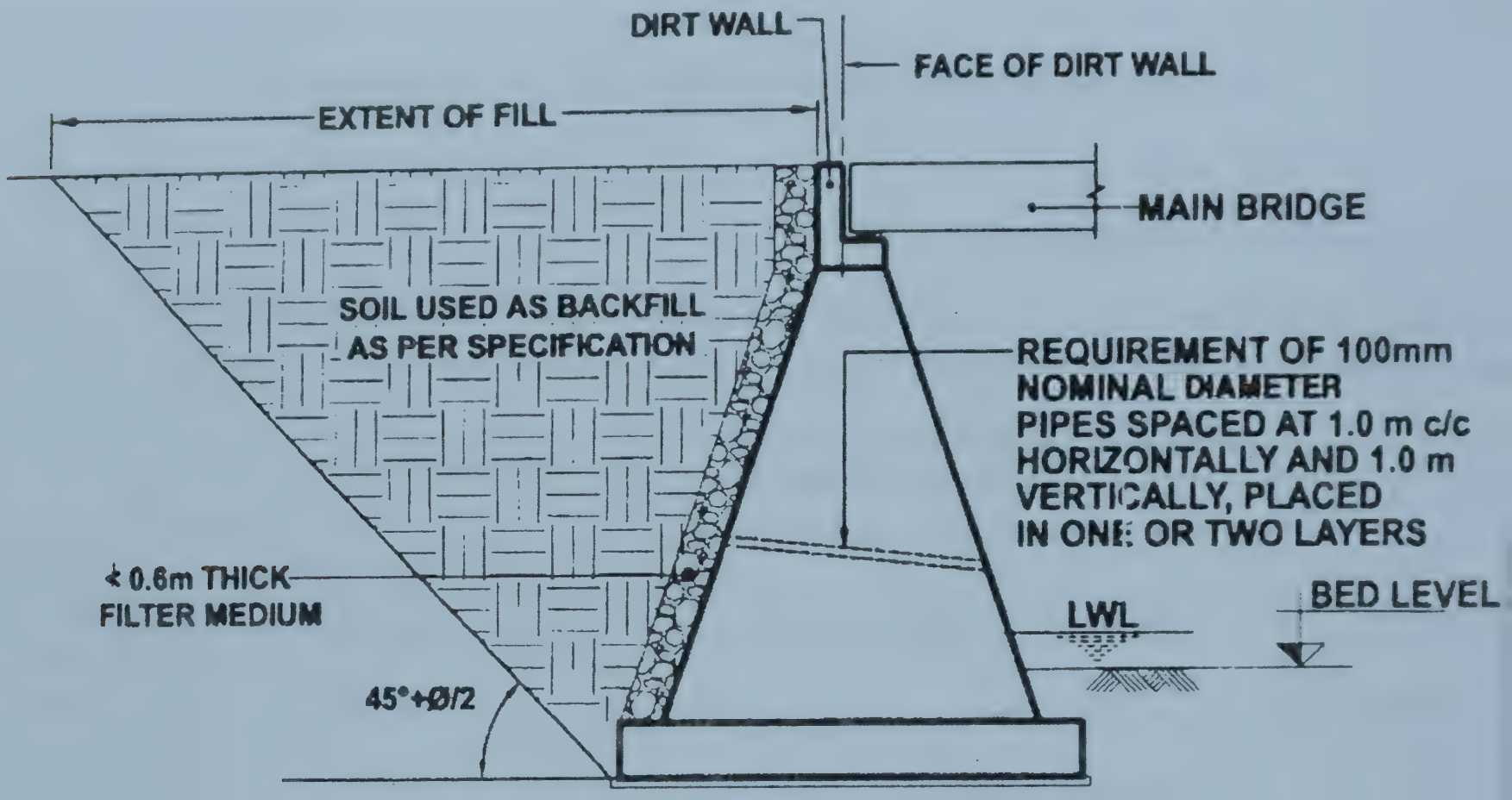


Fig. 1

Notes:

1. Active wedge of soil mobilized in developing active pressure has to be filled by selected earth
2. In case, projection of footing towards earth-fill is less than 600 mm, the filter shall be supported near the top of the footing

4. PRECAUTIONS TO BE TAKEN DURING CONSTRUCTION

4.1 The sequence of filling behind abutments, wing walls and return walls shall be so controlled that the assumptions made in the design are fulfilled and they should clearly be indicated in the relevant drawings. For example, if the earth pressure in front of the abutment is assumed in the design, the front filling shall ;also be done simultaneously along with the filling behind abutment, layer by, and in case the filling behind abutment should also be deferred to a later date. In case of tie beams and friction slabs, special care shall be taken in compacting the layer underneath and above them so that no damage is done to them by mechanical equipment.

4.2 Special precautions should be taken to prevent any wedging action against structures, and the slopes bounding the excavation for the structure shall be stepped or strutted to prevent such wedging action.

4.3 Adequate number of weep holes not exceeding one metre spacing in both directions should be provided to prevent any accumulation of water and building up of hydrostatic pressure behind the walls. The weep holes should be provided above the low water level.

Appendix-9

(Clause 703.2.2.2)

GUIDELINES FOR CALCULATING SILT FACTOR FOR BED MATERIAL CONSISTING OF CLAY

In absence of my formula 'K_{sf}' may be determined as per **Clause 703.2.2** and may be adopted based on site information and behaviour history of any existing structure. The clayey bed having weighted diameter normally less than 0.04 mm offers more resistance scour than sand though mean depth of scour as per the formula given in **Clause 703.2** indicates more scour. In absence of any accepted rational formula or any data of scour at the site of the proposed bridge; the following theoretical calculation may b adopted:

i. In case of soil having ø <15° and c (cohesion of soil) > 0.2 kg/cm^2

'K_{sf}' calculated as follows:

$K_{sf} = F(1+ \sqrt{c})$

Where

F = 1.50 for ø> 10° and < 15°

= 1.75 for ø> 5° and 10°

= 2.00 for ø< 5°

ii. Soils having ø >15will be treated as saandy soil even if c is more than 2N/mm^2 and silt factor will be as per provisions of **Clause 703.2.2**.

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the Code/Guidelines/Manual, etc. from the date specified therein)

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